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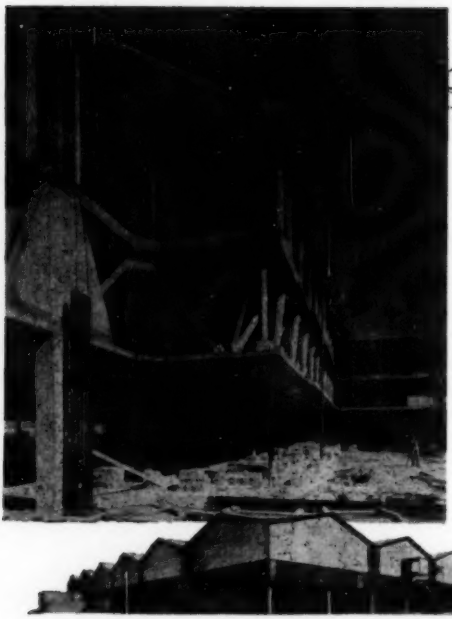
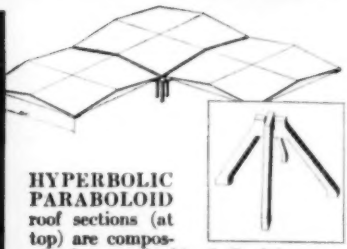
ACI PROCEEDINGS VOL. 55

SPACE PLATFORM





HYPERBOLIC PARABOLOID roof sections (at top) are composed of warped thin-shell slabs, cast in place in reusable forms. Slender supporting columns form hollow squares which contain roof drainage and air-conditioning ducts. V-TETRAPODS (above right) are four-footed diagonal supports for space frame. Precast of reinforced concrete at the site, they are welded in position between first floor ceiling and floor slab above, making a rigid unit through which conditioned air and utilities are fed to all areas of the plant. Concrete met specified strengths up to 5,000 psi.

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No. 9

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PROCEEDINGS VOL. 55

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DOUGLAS MCHENRY, retiring Institute president, comments on the concrete industry and reasons why it need not take a back seat to other fields of science and technology. He also points out some industry shortcomings.

•

Coauthors J. D. PIPER and WALTER H. PRICE report on "Russian Progress in Concrete Technology" as observed on their 1958 trip to Russia as members of a delegation invited to inspect concrete construction in Moscow and Leningrad.

•

"Shear Strength of Two-Span Continuous Reinforced Concrete Beams" by JOSE J. RODRIGUEZ, A. C. BIANCHINI, I. M. VIEST, and CLYDE E. KESLER reveals results of an intensive investigation conducted to determine the effect of continuity on the shear strength of statically indeterminate beams.

•

J. F. CAMELLERIE presents a brief description of the slip-form construction process, its advantages, and limitations in his paper "Slip Form Details and Techniques."

•

HAROLD S. DAVIS and ORVILLE E. BORGE offer data describing the physical properties of a high-density concrete tested under standard conditions and after heating at temperatures of 85, 200, and 300 C in their joint contribution "High-Density Concrete Made with Hydrous-Iron Aggregates."

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

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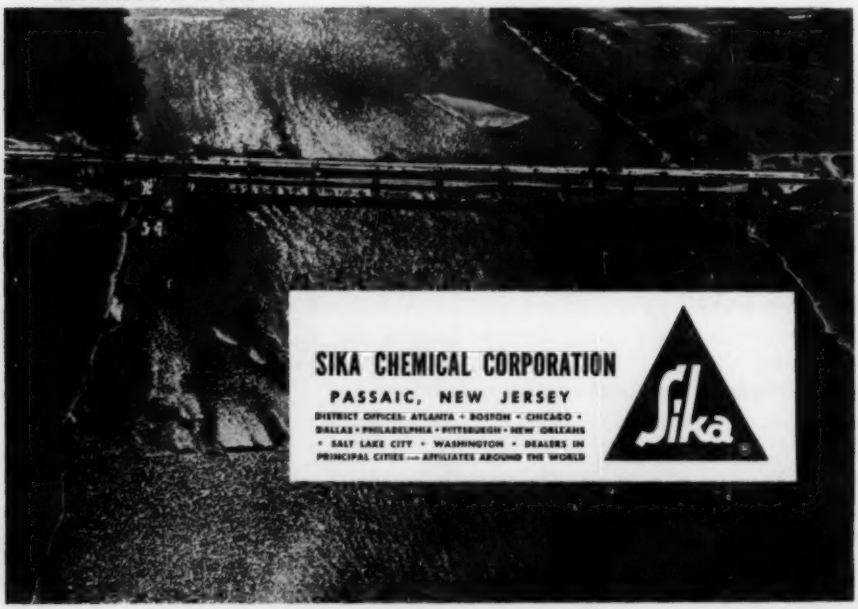
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Title No. 55-59

Milestones of technology as reviewed at the dedication of the ACI Headquarters Building

Commentary on Concrete

By ROBERT F. LEGGET

■ THIS INTERNATIONAL gathering and the official opening of the new ACI building surely mark a milestone in the long history of the technology of concrete. The building that has now been well and truly dedicated, for the use of the American Concrete Institute in its wide-spread services, is of unusual interest in the architectural sense, unique indeed in some of its features. Is it not the first building ever to be erected by a professional and independent society whose aim is the advancement of the state of the art of concrete technology? In many countries, including the United States, there exist splendid organizations devoted to the promotion of the proper production and use of portland cement and concrete that are supported financially (at least in part) by the manufacturers of cement. Their work is of great value. The American Concrete Institute is, however, to the best of my knowledge, unique in this field in that it is supported entirely by its individual members who work together not only for the advancement of the art but also for the promulgation of codes of good practice—in design and construction—which they make freely available for voluntary use.

This is a fine thing, one of the great characteristics of the engineering scene in North America, now symbolized for this society by the new building, itself paid for in large part by the members themselves. The very freedom of enterprise which it betokens is a continuing responsibility, but of this there is no need for me to speak. The address of your President is in itself warrant enough for full confidence that the American Concrete Institute is going forward to still finer work, fully conscious of these responsibilities, well aware of the international leadership that is its heritage and its challenge.

MILESTONES OF CONCRETE TECHNOLOGY

Milestones are markers of a quieter age than ours. Is it not significant that the great new superhighways of today rarely include such useful markers of the passing miles? We can, however, still speak of milestones metaphorically, and this is certainly an occasion for using this happy simile. Do you recall the old rhyme about the milestone . . .

"It tells two things well worth the knowing;
Where we are and where we're going;
More candid than that other stone,
Which only says that we have gone."

May we, together, spend a few minutes in seeing where we are and where we're going with this wonder material concrete?

Concrete, as we know it today, has a history of about a century—a century of remarkable advance in its use, as in all branches of technology. It is easy to become so enthusiastic about the things now done with concrete that it may be salutary to remind ourselves first that although we, as engineers, know what we are talking about when we discuss concrete, the man in the street generally does not—even after a century of such splendid technical progress.

How often do we hear, and even read, of an operation described as "pouring cement." In a fine book which I have just finished reading, a stimulating discussion of work and contemplation by one of your eminent philosophers,[1] I came across this expression twice—and this in the writing of a learned student of manual work. How often do we hear of "cement sidewalks"? Why is it, I wonder, that the editors of even our greatest newspapers and of our leading journals of opinion, expert as they are, have not yet discovered that cement is but one of the ingredients of concrete? And even as engineers, we need not be too complacent. How many engineers still invariably talk of "pouring" concrete when they know quite well that, if the concrete they use is good concrete, it can not be poured but must be placed? Even so eminent a journal as *Engineering News-Record*, which tries to follow a rule of using "place" or "cast," still misses sometimes and lets "pour" slip in.

Tomorrow's lesson from yesterday

There is still a tremendous educational job to be done—a fact of which I am reminded almost every day, as I pass on my way to work a concrete sidewalk which failed miserably, mainly because it was not properly cured in the

ROBERT F. LEGGET, director, Division of Building Research, National Research Council of Canada, Ottawa, received his MS from the University of Liverpool. He worked for a consulting firm in England before moving to Canada. In Canada, Mr. Legget was engaged in heavy construction and was on the faculties of Queen's University and the University of Toronto. In 1947, he was invited to establish the Division of Building Research for NRCC.

heat of high summer. And you will know, as I know, that the necessity of curing fresh concrete, and the use of damp sand for this purpose, was recognized (by Wilkinson) at least 100 years ago. But these are small things; let us look at the other side of the picture.

What a versatile material is concrete! Has it ever occurred to you that there is no other building material which can be used throughout the wide range from the placing of a few cubic inches around a pole in a garden to the use of over 10,000,000 cu yd in a single man-made structure such as the Grand Coulee Dam. It can be made by a child; it can be produced by the ton. It can be mixed on a shovel; it can be manufactured in majestic fully-automatic plants. It can be placed in tropical climes, cooled if necessary as it sets; I have personally supervised the placing of concrete when the air temperature was −40 F, and concrete was heated, instead of being cooled, until it gained its full stability. It is man's equivalent of nature's conglomerates, with the added advantage that man can choose his constituent materials and so even get the better of nature—provided that he selects his materials carefully.

AESTHETIC POSSIBILITIES

More than this, of all the more common structural materials, concrete is the only one which is completely susceptible in form and size to the molding hand of the designer. Despite all that is sometimes said to the contrary, engineers in general are appreciative of true aesthetics. What possibilities concrete places in the hands of the master designer for combining beauty with utility! It has been well said that "beauty is a refinement on function which induces visual comfort or stimulates visual delight or visual excitement." With how little extra effort can concrete structures be charged with delight? And how good it is to think that more and more the aesthetic possibilities of concrete structures are being appreciated by designers—engineers, and engineers working together with architects in happy combinations that can remind us of the master builders of another age. We can think together of the clean-cut lines of some of your TVA dams and power houses, the gaunt outlines of Canadian grain elevators—bastions of the great plains, the slender arched bridges of Robert Maillart in the remote mountain valleys of Switzerland, the splendid symmetry of some of the more recent works of Nervi in Italy with their almost unbelievable web-like structures. These and countless more examples from almost every country testify to the skill which the engineer has acquired in molding this most natural of all artificial materials to his designs, for the use and for the delight of his fellow men.

PROGRESS ABROAD

I speak of countries overseas. You know well the concrete works of your country and of mine. Would you care to glance briefly at some concrete structures in the far places of the world, places which it has been my great privilege

to visit in recent years on official duty? Your imaginations are vivid; let them provide you with a magic carpet to circle the globe from this pleasant room in the busy heart of Detroit. Flying off to the west, and spanning the broad Pacific, we can recall the vast quantities of concrete made in recent years using coral for aggregate, now to be seen on many a tropical atoll. Right at the other side of the world, we come to New Zealand—a small country with a population of not much more than 2,000,000, but with truly significant engineering works worthy of a population several times as large. As we fly in over Auckland, at the extremity of the North Island, we shall see one of the pioneer reinforced concrete bridges of the world, the Grafton Bridge which, when built in 1910, was the longest single span reinforced concrete bridge in the world.[2] Although the main arch with its span of 320 ft has long since been exceeded in size by other structures, this pioneer structure yet serves its city well and has a long life still ahead of it. It does not take long for a visitor to New Zealand to find that this is a country where earthquake shocks are an ever present danger. As engineers, we shall notice how satisfactorily structures of reinforced concrete have performed even when subject to such severe loading, the monolithic character of concrete structures giving them particular advantage for all such critical locations.

Down under

Crossing the Tasman Sea, we come to the Island continent of Australia. There are concrete structures aplenty to be seen in Sydney but its skyline is still dominated by its famous steel arch bridge. If present plans are carried out, however, a rival will soon appear in the form of an opera house, the architecture of which will be even more "functional" than that of the ACI building, the architect having attempted to reproduce in a concrete structure the appearance of sails in the wind, so widespread and lovely a sight on the blue waters of the harbor. It is in Melbourne, however, that we shall find the most notable concrete structure of Australia, this being the great dome of the public library. When built, it was the largest reinforced concrete dome in the world; those older members who recall the 1914 war may find interest in the fact that it was designed by Sir John Monash who achieved international fame as the leader of the Australian forces in that first world conflict.[3]

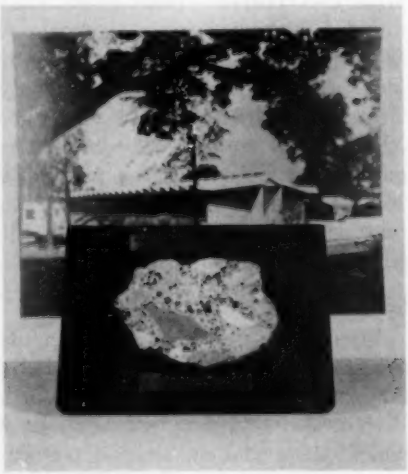
We shall have to cross to the quiet island of Tasmania, however, to see quite the most unusual concrete structure of Australasia, the arched bridge across the harbor of the beautiful capital city in Hobart. This is a reinforced concrete bridge arched in plan, and not in profile, for it is also a floating bridge. Designed by a good friend of mine, David Isaacs, it was constructed in 1936. Foundation conditions dictated the unusual nature of the structure which is 3156 ft long, and which weighs about 25,000 tons. The tidal range is 9 ft and strong ocean swells add to the complications of the site with the result that the bridge sometimes acts as an arch in compression, sometimes as a catenary in tension, depending on winds and tides.[4]

Singapore

Singapore must be our next stop, that amazing little island almost exactly on the equator. Its area is only 224 sq miles and yet about 1,500,000 people live on this famous island, most of them in such crowded quarters that these have to be seen to be believed. But for almost 30 years the Singapore Improvement Trust has been making steady progress at improving housing conditions. Its most recent housing developments constitute some of the finest reinforced concrete structures which it has been my privilege to see anywhere. Can you visualize clean-cut 14-story blocks of simple flats, constructed entirely of reinforced concrete, with flat roofs, open balconies, a judicious use of bright colors, and pure white precast concrete grillages over all open wells? These are exciting buildings, exciting aesthetically, and exciting to watch under construction, for the "jungle" scaffolding which is used right up to the top of the full 14 stories consists of slender sticks of bamboo merely lashed together.[5]

India

On to India, that amazing sub-continent with its population of almost 400,000,000, a population increasing at the rate of 5,000,000 every year. These fantastic numbers will make you think of the traditional type of Indian building, seen in typical pictures of Indian villages. Simple brick and soil structures still dominate the national scene but India is awakening. The fine pioneer engineering work of the British is now being continued at an ever increasing rate by the engineers of the new nation. Would it surprise you to know that the first prestressed concrete bridges in the world, designed for railway use, were constructed in India? Exceptions these may have been but they were built for the Assam Railway in 1949.[6] As early as 1934,



Old and new—A piece of Roman "concrete" with the new ACI Headquarters Building in the background. The Roman "concrete" was presented to ACI by the Division of Building Research, National Research Council of Canada at the dedication of the ACI building. The inscriptions read:

ROMAN 'CONCRETE' FROM NEAR ONE OF NERO'S AQUEDUCTS: VIA VERGINE A. D. 60

A TOKEN OF WELL-WISHING: 29.X.1958 DBR, NRC, CANADA, TO ACI

a mass concrete dam (the Methur Dam in south India) 5300 ft long and 200 ft high had been built across the Cauvery River. And today there is being constructed of mass concrete, with much American assistance and with modern construction methods, the great Bhakra Dam 680 ft high and 1700 ft long, across the Sutlej River, for what will be one of the largest multi-purpose projects of the world.

Not all Indian concrete structures follow the lead of the West, however. In New Delhi one can see a beautiful temple in the ancient Hindu style of architecture. One can admire it, and then be surprised to find that it was built as recently as 1939 in reinforced concrete—a truly adaptable material. And most of the dams being built in India are still constructed by hand, of masonry hand-placed in cement mortar, up to heights far exceeding 100 ft. The great manpower problem of India is vividly shown by the fact that this type of construction is still, in 1958, about two thirds of the cost of mass concrete placed with the aid of all the most modern equipment.[7] If time permitted, we could inquire about the ancient use on the Malabar Coast of mortars made of lime mixed with molasses (a clue for modern research workers?). Would, too, that we had time to fly across the Indian Ocean to the west to visit South Africa, another country which uses concrete well. Our travel must, however, be to Europe.

Europe

Many of you will know the countries of western Europe where so much splendid concrete work is to be seen. How pleasant it would be to wander in France, to visit the sites of some of the early pioneer reinforced concrete structures, certainly to make pilgrimage to le Pont d'Ivry where in 1903—55 years ago, Consideré tested a large reinforced concrete bowstring arched bridge to destruction.[8] We could find much of interest in Belgium, in Holland, and in West Germany. So much fine work has been done in all these countries, especially in recent years, that it would be almost invidious to select any for individual mention apart, perhaps, from Professor Paduart's superb "arrow" at the Brussels Exhibition. Research into concrete problems has also been actively pursued, notably at the fine Materialprufungsamt der Technische Hochschule in Munich. In Switzerland, we would be astonished by the audacity of some Swiss concrete bridges, even as we would be enthralled by the beauty of many of them in their superb natural settings. In Denmark we could walk over a small reinforced concrete bridge that was built in the year 1894 and which still gives good service, crossing a main road with a span of just over 70 ft.

Scandinavia

Flying north into Scandinavia, we come to another area where modern reinforced concrete work is indeed challenging. We would see precast concrete units in use on a scale equalled only, perhaps, in the USSR, and possibly in Britain and France. In Stockholm I have visited a real concrete "factory" which made and distributed precast, prestressed concrete units not just in

that city but to all parts of Sweden. In that same lovely city, so well called the "Venice of the north," we would see reinforced concrete arched bridges that are masterpieces of design, with spans amongst the longest in the world. (I hesitate to mention which Swedish bridge has the greatest span since I found this still to be a matter of argument among Swedish friends.) We would find, too, another of the great concrete research centers of the world, the Swedish Cement and Concrete Research Laboratory.

Far north

I must ask you, however, to travel still further north with me, over first to Finland—that valiant little land—self-sufficient for many years in its cement needs, and using concrete with vigor and vision. On still to the north, however, right into Lapland, to Ivalo and to Karasgniemi at the far northern tip of Finnish Lapland. Concrete in Lapland? Yes, here too. Just over a year ago, I stood by a concrete mixer in Ivalo operated by a Laplander and saw most excellent concrete being placed for building foundations in this land of the midnight sun. But even I was not prepared for what I found when I went further north still, across the border into Norway, and on up to Hammerfest—the most northern town in the world, located beyond the 70th parallel, only a thousand miles from the North Pole. This ancient fishing port with a hardy population of about 5000, was one of the Scandinavian towns to be devastated in the shocking waste of war, every single building razed to the ground by fire in 1944. Its rebuilding was virtually complete last year, the new town hall being one of the last of the new buildings to be completed. And the town hall, as also many of the larger buildings of the town, was a first class piece of reinforced concrete construction.

WHERE ARE WE GOING?

From the Antipodes to the Equator, from the Equator to far north of the Arctic Circle, concrete well and truly used all the way. Where better, in imagination, could we ask our second question—"Where are we going?" With concrete already used in such a variety of ways, it would not be reasonable to suggest that any major changes in concrete technology or in the use of concrete will occur in the foreseeable future. The variability of concrete ingredients can never be forgotten, requiring ceaseless vigilance for the detection of any new and previously unknown reactions, constant research into the problems raised by the use of new and untried materials. The success of air-entraining agents suggests, in an almost tantalizing way, that there may yet be found an admixture which really would influence the behavior of concrete (this statement having no relation to, nor reflection on admixtures now on the market).

Design

Turning to design, surely the time has come when the problem of designing satisfactorily for shear stresses must be solved? Working stresses must be continually under review, as efforts are made to raise them effectively, per-

haps some logic introduced into their determination. Even more important, possibly, will be the more mundane task of really designing "concrete for permanence." In this, careful reviews of experience with concrete structures in practice will always be valuable, such as American studies of the performance of concrete in dams, and the British report on the durability of concrete buildings.[10] Recent British practice supports one further prediction—that we shall see a considerable increase in the use of precast concrete units. In London, there was recently finished a 10-story concrete office building, completely precast, typical of recent European advances in this direction.[11]

But with this briefest of references to British practice, we should head to the west again, to complete our imaginary journey. As our magic carpet carries us away from England's green and pleasant land, we may catch sight of a solitary lighthouse, seemingly rising out of the sea, south of Plymouth. This is the Eddystone Lighthouse. John Smeaton was entrusted with its reconstruction in 1756, just two centuries ago. This first of civil engineers was a true research worker; his investigations for the final mixture of blue lias lime and pozzolan from Civita Vecchia laying the foundations for all modern cement and concrete research. And where did Smeaton get his inspiration? From the Romans, and from the records of their works of 1700 years before, works which were so strangely forgotten throughout the middle ages. Let us, also take a quick glance back in time across the centuries for was it not Edmund Burke who said "people will not look forward to posterity who never look backward to their ancestors."

Early uses

What interest can we find. Here, for example, is Pliny writing in his *Natural History*: "Cisterns should be made of five parts of pure gravelly sand, two of the very strongest quicklime, and fragments of silex (lava) . . . when thus incorporated, the bottom and sides should be well beaten with iron rammers." And then we think that vibrated concrete is something new! Admittedly, the Romans did not use portland cement but their mixtures of aggregate, lime, and pozzolan gave a material that is the direct ancestor of good concrete of today. They even used lightweight aggregate, the magnificent dome of the Pantheon, with a clear span of 142 ft 6 in., being possible only because of the use of pumice as aggregate. They used wooden molds just as we do today. And we know that they really did "pour" some of their "concrete" instead of placing it, as they should have done, by what we can see of its appearance today, 2000 years later. Sad to relate, they did not always clean out their formwork as they should have done, sometimes leaving chips of wood from the forms to be mixed in with the "concrete."

You will think that I exaggerate. But if we could go together to the Flavian Palace on the Palatine in Rome, built more than 2000 years ago, we could see in the foundation walls the marks of the boards then used for formwork, clear indication of too much water in the mix. And the chips of wood in the forms? These must be seen to be believed. So much do we owe to the Romans,

and so useful is it to be reminded occasionally that all engineering is not the work of the twentieth century that I would like to conclude this brief commentary by presenting to the President of the Institute as a small token a piece of Roman "concrete," almost 2000 years old, fairly well graded but containing a small chip of wood still clearly to be seen.

If, therefore, Mr. President, this small specimen can find a place in the new building, will you please accept it not only as a token of well-wishing to your Institute from Canadian engineers and research workers, coming to you across the borders of space, but coming also across the borders of time, as a reminder of our debt to the past, a symbolic token of all that has been done down the ages in developing the concrete technology of today, challenge indeed that we must never forget the ancient words:

> "That which our Fathers bequeathed to us, we must earn to possess."

SPECIAL ACKNOWLEDGMENT

The specimen of Roman "concrete" was kindly obtained through the Canadian Embassy in Rome (N. F. H. Berlis, Chargé d'Affaires) with the expert and interested assistance of Professor Maurizo Borda, vice-director, Museo Nazionale Romano, Rome, Italy.

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Title No. 55-60 . . . Method for determining the influence of grade of reinforcing steel on blast resistance of beams is based on preselected criteria of failure. The peak dynamic load capacity is calculated for beams of a given configuration but having different percentages and grades of reinforcing steel. These calculations are made for blast-type loads of different duration. Plots of the results indicate that suitability of various steel grades will depend primarily on maximum permissible deflection, characteristics of the loading, and amount of tension steel used.

Blast Resistance of Reinforced Concrete Beams Influenced by Grade of Steel

By WARREN A. SHAW and J. R. ALLGOOD

IN THE DESIGN of a reinforced concrete beam to resist blast loading, the engineer must choose the grade of steel to be used. The grade of steel in a given beam determines the static load-deflection characteristics which in turn influence the dynamic load capacity of the member. The question is, "What grade of steel, or what resistance characteristics of a beam, are best for withstanding a given blast loading?"

Several authors have treated the problem[1,2,3] both from an experimental and a theoretical point of view. Recent discussions,[4] however, indicate some lack of agreement on the subject and suggest that a method is needed for comparing dynamically loaded beams reinforced with steels of different stress-strain properties.

BASIS FOR EVALUATION

In most blast resistant construction the design objective is a structure whose deflection will be less than some specified limit under the design loading. Behavior of the structure or structural element will be different depending on the duration of the pressure wave. The distinction between a short

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and long duration load in a given case cannot be precisely stated without knowing the complete load-time history; however, loads having durations greater than six times the natural period of the beam are usually regarded as long duration loads.

A structure which can deflect plastically without loss of resistance will allow time for a short duration pressure wave to decrease in magnitude to a value which the structure can support. Thus, the peak dynamic load capacity may be several times the static yield load.[5]

For long duration loads, the advantage of permitting plastic deformation is not so marked as for short duration loads; the capacity is only increased about 10 percent[6] above static yield load. Nonetheless, designing structural members to deflect plastically is usually desirable to avoid sudden brittle-type failure. It remains to establish how much ductility is required and how it is best obtained.

For reinforced concrete beams to withstand large plastic deformations at least four requirements should be met:

1. They should be underreinforced (that is, the yield point of the tension reinforcing steel should be reached before the ultimate compressive strain of the concrete or compression steel is developed).
2. The reinforcing steel must be sufficiently ductile to permit the beam to retain its resistance through large deformations.
3. Compression steel should be used. The yield and ultimate deflection of beams without compression steel cannot be accurately predicted. Further, properly tied compression steel permits the member to sustain load even after compressive failure of the concrete occurs.
4. The member must be designed to avoid the development of connection failure, brittle failure, shear slides, and bond failure.

The essence of these requirements is that a beam must have sufficient yield strength and sufficient ductility to resist the imposed blast load. For a given member with a fixed percentage of reinforcement, hard grade steel will provide greater yield strength than structural or intermediate grade steel; however, structural grade steel will provide greater ductility. The question is—which of these factors is more important?

Before this question can be answered, a basis for comparison of beam behavior must be established. The writers have selected two criteria. First

is that the beams will be allowed to deflect to their ultimate deflection (where ultimate deflection is defined as the deflection at compressive failure of the concrete). The second criterion is that the maximum deflection shall not exceed 1/50 of the span. This value of limit deflection is selected arbitrarily. Using these criteria, beams of identical dimensions reinforced with structural grade or hard grade steel are compared by computing the peak load capacity for a prescribed ratio of load duration to natural period of the member as a function of the percentage of tensile steel. Plots of such calculations provide a means of comparing beams reinforced with different grades of steel subject to the different criteria of failure.

DATA AND THEORY USED IN COMPARISON

It is convenient to use available test data[1] which give the static load-deflection characteristics of beams reinforced with structural grade or rail steel in the amounts of 0.84, 1.68, and 2.52 percent. These tests were on model-size beams with a span of 6 ft. The yield deflection and other important properties of the beams are presented in Table 1.

Properties of materials used in the beams were: concrete strength, 3500 psi; static lower yield point of the steel, 44,000 psi and 57,000 psi, respectively, for the structural grade and rail steels. Although Reference 1 describes the lower strength steel as structural grade, the yield point satisfies the requirements for intermediate grade steel. From tests of Jackson and Moreland,[7] it is found that the probability is approximately 14 percent that structural steel A-7 would have a yield point equal to or greater than 44,000 psi. It is expected that approximately the same percentage would apply to structural grade reinforcing steel.

Idealized, experimental, static load-deflection curves for the beams are shown in Fig. 1, 2, and 3. (These curves could have been obtained equally well by theoretical procedures.[8]) The curves are idealized to the extent that distinct values of yield load and deflection are shown. The experimental behavior of the beams did not exhibit a distinct knee in the load-deflection curve so the elastic and plastic range slopes were extrapolated to locate this hypothetical point. It is emphasized that the resistance curves of Fig. 1, 2, and 3 are of principal concern in the remainder of the discussion. The material properties are only important insofar as they govern the resistance curves for a particular beam.

TABLE 1—STATIC PROPERTIES OF BEAMS

Percent tensile steel, p	Rail steel			Structural grade steel		
	Yield load, lb	Yield deflection, in.	Ultimate deflection (y_m), in.	Yield load, lb	Yield deflection, in.	Ultimate deflection (y_m), in.
0.84	2140	0.55	1.75	1260	0.21	3.15
1.68	3700	0.70	1.36	2440	0.38	3.00
2.52	4890	0.91	1.12	3200	0.50	2.64

With the chosen static test data and available design curves,[5,9] the response to a load with an instantaneous rise and a triangular or step-time variation is readily obtainable. The cited design curves are based on the spring-mass analogy which has been experimentally justified.[6,10] They give the ratio of applied dynamic load to dynamic yield load, P_m/Q_b, as a function of the ratio of maximum dynamic deflection to dynamic yield deflection, y_m/y_b, for a given ratio of pulse duration to natural period, t_1/T. This variation for a step load and for ratios of pulse duration to natural period of 0.5, 2.0, and 5.0 is plotted in Fig. 4 and 5.

Before either of the two previously stipulated criteria can be used, it is necessary to account for the increase in yield point of the tension steel caused by the rapid application of load. The few available tests show that the increase in yield point with strain rate is slightly less for hard grade steel than for structural grade steel.[6] It is assumed here, however, that both grades of steel experience a 35 percent increase in yield point. Using this increase, Q_b and y_b (the dynamic yield load and deflection) are calculated from the corresponding static values taken from Fig. 1, 2, and 3.

Using the first criterion, which permits the beams to deflect to their ultimate deflection, the ratio of ultimate deflection to dynamic yield deflection is computed. Then, with the aid of Fig. 4 and 5, the ratio of applied load to dynamic yield load is determined for different ratios of pulse duration to natural period. It is now a simple matter to calculate the least value of applied dynamic load which will produce ultimate deflection. Plots of the variations of applied load with percentage of steel reinforcement are presented in Fig. 6, 7, 8, and 9. The portion of these curves with small dashes indicates regions where an extrapolation of the beam response curves was necessary.

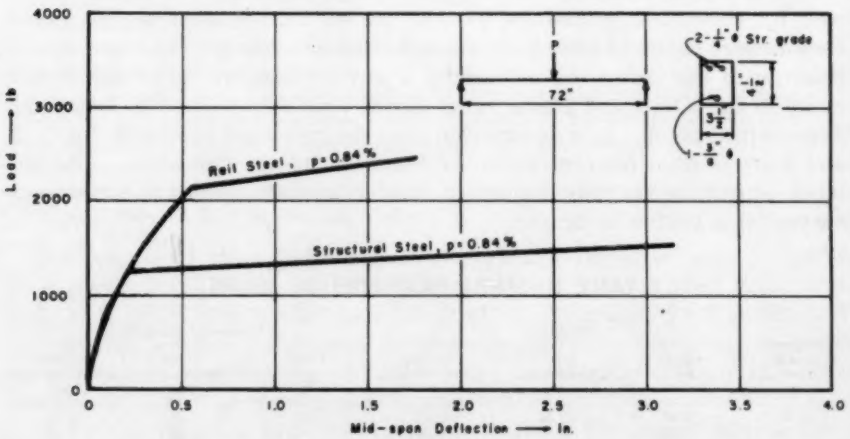

Fig. 1—Idealized static load-deflection curves. p = 0.84 percent

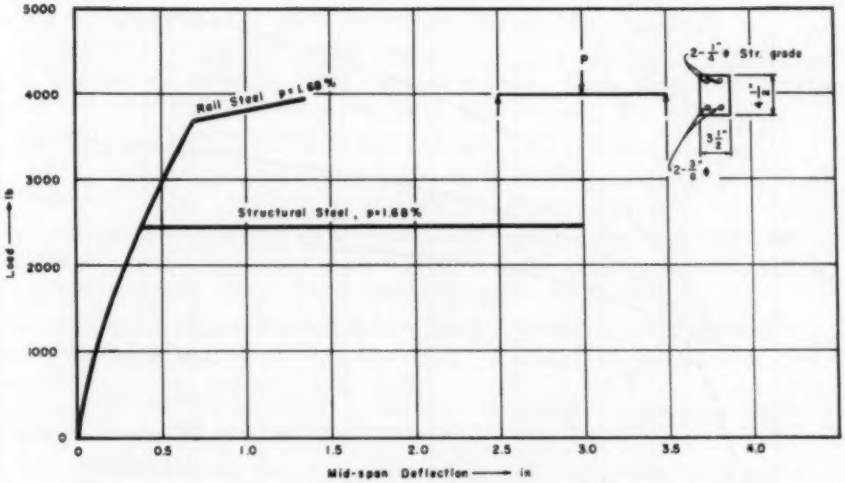

Fig. 2—Idealized static load-deflection curves. $p = 1.68$ percent

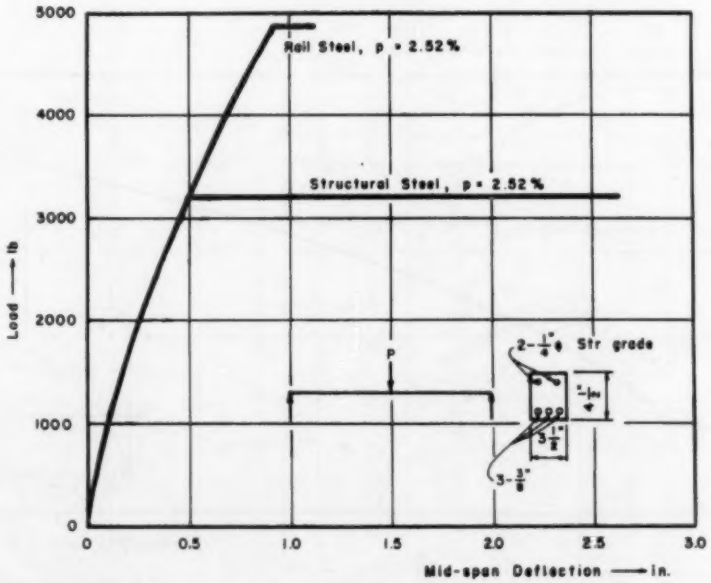

Fig. 3—Idealized static load-deflection curves. $p = 2.52$ percent

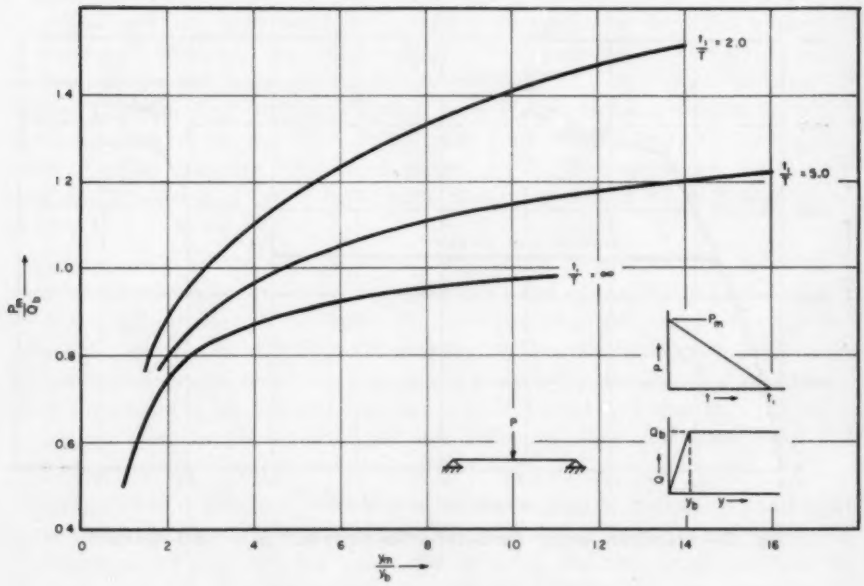

Fig. 4—Beam response to dynamic load

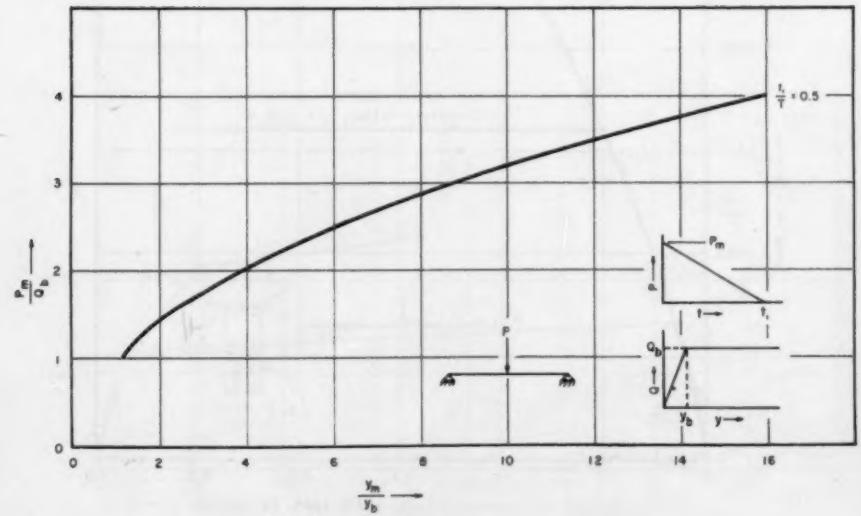

Fig. 5—Beam response to dynamic load

It is important to recognize that the *initial* maximum deflection is not necessarily the maximum deflection which a member will experience. If the dynamically applied load is sustained after the initial response, the beam will seek static equilibrium. For loads greater than the static ultimate load the member will continue deflecting until it fails. Thus, for a long duration load, the peak dynamic load is the static ultimate load. For most beams this load is about 10 percent greater than the static yield load.

In Fig. 9, for simplicity, the ultimate static load is taken as equal in magnitude to the static yield load. In this case the peak load capacity for all members with structural grade steel is governed by the static yield load. But, for the beams containing more than 1.25 percent rail steel, the peak load capacity is governed by the relatively limited ductility of these members.

A similar calculation has been made using the criterion that the maximum deflection shall not exceed the limit deflection of 1/50 of the span. The results are also given in Fig. 6-9.

Numerical example of procedure

It is desired to determine the peak dynamic load capacity for a beam reinforced with structural grade steel (p = 1.68 percent) when subjected to a triangular shaped load with t_1/T equal to 5.0. The static load-deflection characteristics for this beam are shown in Fig. 2. From this figure, the following values are noted: yield load = 2440 lb; yield deflection = 0.38 in.; and ultimate deflection = 3.00 in. It is assumed that the steel will experience a 35 percent increase in yield point as caused by the rapid loading of the beam; hence, the values of dynamic yield load Q_b and deflection y_b are 3290 lb and 0.513 in., respectively.

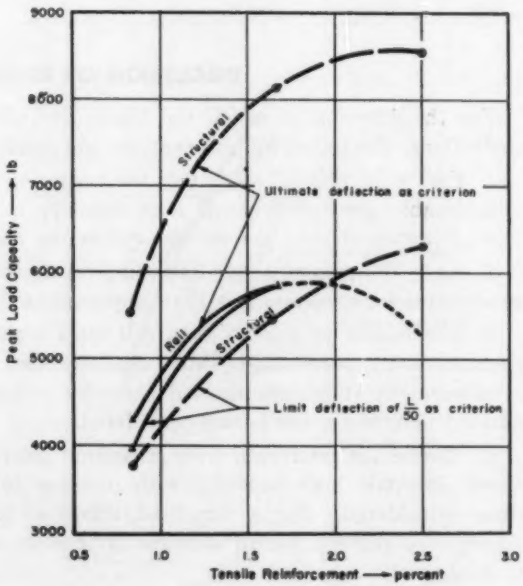

Fig. 6—Dynamic capacity of beams $\left[\frac{t_1}{T} = 0.5\right]$

The ratio of ultimate deflection to dynamic yield deflection then is

$$\frac{y_m}{y_b} = \frac{3.00}{0.513} = 5.85$$

From Fig. 4, for $t_1/T = 5.0$ and $y_m/y_b = 5.85$, the ratio of applied dynamic load to dynamic yield load is

$$\frac{P_m}{Q_b} = 1.05$$

and the peak dynamic load which the beam can sustain without exceeding the ultimate deflection is

$$P_m = 1.05\ (3290) = 3450 \text{ lb}$$

Therefore, this particular beam when subjected to a load of 3450 lb will deflect 3.00 in.

Now assume that it is desired to limit the deflection of the beam to $L/50$ or 1.44 in. The values of dynamic yield load and deflection of the beam are the same as before, and the ratio of allowable maximum deflection to yield deflection becomes

$$\frac{y_m}{y_b} = \frac{1.44}{0.513} = 2.81$$

From Fig. 4, for $t_1/T = 5.0$,

$$\frac{P_m}{Q_b} = 0.89$$

and $P_m = 0.89\ (3290) = 2930$ lb.

DISCUSSION OF RESULTS

For the criterion in which the beams are allowed to reach their ultimate deflections, the following observations are made:

1. For ratios of t_1/T equal 0.5, the beams with structural grade steel have considerably greater dynamic load capacity than the beams with rail steel (for all values of steel percentage within the range considered).

2. As t_1/T increases above 2.0, rail steel shows a superiority over structural grade steel for amounts less than approximately 1.7 percent.

3. The beams reinforced with rail steel reach a maximum dynamic load capacity as the percentage of steel reinforcement is increased. This amount of reinforcement at maximum load capacity varies between approximately 1.8 and 2.4 percent for the beams considered.

4. The beams reinforced with structural grade steel continue to gain additional dynamic load capacity with increase in steel percentage within the range considered. For a step load, this load capacity is increased approximately 130 percent by an increase in tension steel reinforcement from 0.84 to 2.52 percent.

If the criterion of limiting the deflection to a value of 1/50 of the span is selected, the following observations can be made:

1. The ultimate static deflection of the beams reinforced with structural grade steel was between $L/23$ and $L/27$. By restricting the dynamic deflection to $L/50$, the allowable load applied to the beams reinforced with structural grade steel is considerably reduced, particularly for the short duration loads (see Fig. 6–9). By contrast, the maximum reduction in peak load capacity for beams reinforced with rail steel was in no case more than 8 percent.

2. The ultimate static deflection of the beams reinforced with rail steel was less than the arbitrary limit deflection ($L/50$) except for the beams with 0.84 percent steel; therefore, the ultimate deflection was, of necessity, the criterion used for beams with 1.68 and 2.52 percent steel.

3. The beams reinforced with structural grade steel continue to gain additional dynamic load capacity with increase in steel percentage within the range considered.

4. Under the criterion $y_m \leqq L/50$ and for the beams used, those reinforced with rail steel have larger allowable dynamic loads than those reinforced with structural grade steel over nearly the full range of steel percentage and ratio of pulse duration to natural period.

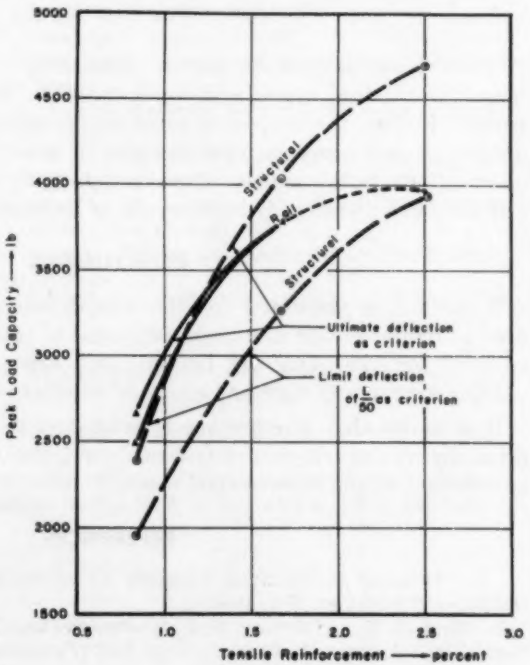

Fig. 7—Dynamic capacity of beams $\left[\frac{t_1}{T} = 2.0\right]$

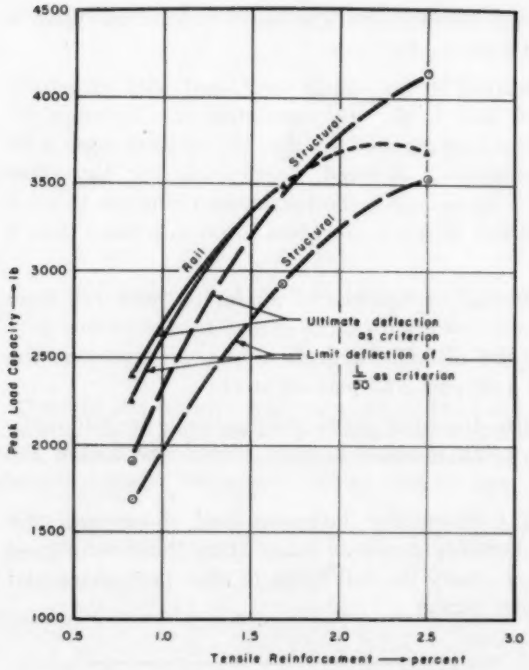

Fig. 8—Dynamic capacity of beams $\left[\frac{t_1}{T} = 5.0\right]$

There is an obvious danger in attempting to generalize these results to beams of all sizes, types, and configurations. Such is not the intent of this paper. Rather, the purpose is to present a method for comparison of various grades of steel used as reinforcement in any given beam or beams. The observations made merely show under what load conditions and failure criteria a given steel may be expected to be superior.

CONCLUSIONS

A method is presented for the comparison of different grades of steels used as reinforcement in a beam subjected to transient loading. This method is useful for comparing the dynamic load capacity of beams reinforced with varying amounts of different grades of reinforcing steel.

It is shown that whether one type of steel is better than another depends primarily on the criterion of failure chosen, the characteristics of the loading, and the amount of tension steel used.

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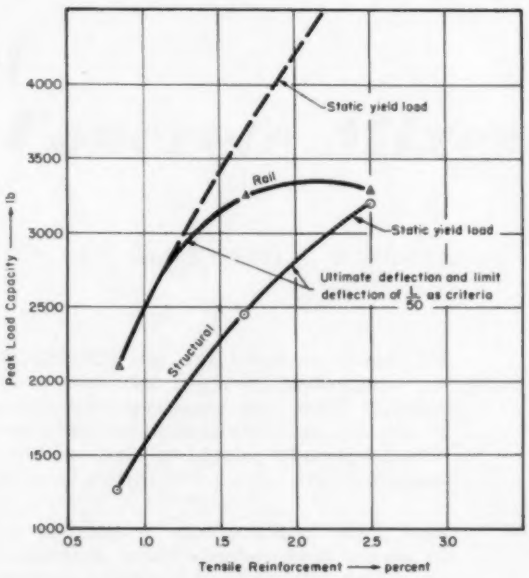
Fig. 9—Dynamic capacity of beams $\left[\frac{t_1}{T} = \infty\right]$



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Title No. 55-61

Control of Concrete Mixes

By EDWARD A. ABDUN-NUR and JOSEPH J. WADDELL

CONCRETE CONTROL PROCEDURES for the Northern Illinois Toll Highway resulted in greater contractor efficiency, lower costs, and closer conformance to specification requirements than usually obtained on similar work. Supervision of the numerous contracts was effected by the section engineers, general consultant, and testing laboratories working in unison to insure quality of materials and uniformity of structural and pavement concrete.

Semiautomatic batching and central mixing plants, which were equipped with automatic recording moisture meters, finish screens for coarse aggregate, and recorders to indicate batch weights, significantly contributed to the good control of concrete quality.

THE CONTROL PROCEDURES and data presented herein describe early operations on the Northern Illinois Toll Highway. The project is a good example of the application of the basic principles presented in "Criteria for Modern Specifications and Control" by Edward A. Abdun-Nur and Lewis H. Tuthill (ACI JOURNAL, V. 30, No. 7, Jan. 1959, pp. 759-768).

From the control point of view, the principal accomplishment of the first season's work was the indoctrination of all working units in the intent of the specifications and how best to attain control. In addition, plants were checked, materials sources approved, mixes adjusted, and sufficient concrete was placed to point out problems and find solutions for them, thus smoothing the operation and control procedures for the subsequent big construction push.

The Northern Illinois Toll Highway, (Fig. 1), is a four- and six-lane divided highway with limited access, extending 187.3 miles. The reinforced concrete pavement is 10 in. thick; together with its bridges, grade separations, and structures, it involves over 2,750,000 cu yd of concrete. This large volume of concrete, divided into four major classes, was placed by many contractors and subcontractors, under the direction of 23 engineering firms acting as section engineers. Attaining uniformity and quality control under such a variety of conditions constituted a major problem.

Edward A. Abdun-Nur, consulting engineer, Denver, served as a consultant on the Northern Illinois Toll Highway where he conducted a materials survey, set up quality control, and developed new standard specifications. Prior to entering private practice, he spent 16 years in various engineering capacities with the U. S. Bureau of Reclamation, Corps of Engineers, and U. S. Indian Service. Currently a member of ACI Committees 214, Evaluation of Results of Strength Tests of Concrete; 611, Inspection of Concrete; and 617, Specifications and Recommended Practice for Concrete Pavements and Bases, Mr. Abdun-Nur has been an ACI member since 1948.

Joseph J. Waddell, chief materials engineer, Knoerle, Graef, Bender, and Associates, Chicago, joined ACI in 1938. He served 18 years with the U. S. Bureau of Reclamation on projects such as Parker, Shasta, and Friant dams, and the Friant-Kern Canal. He was materials engineer for the consultants on the Lake Pontchartrain Bridge. In 1956 he was appointed project materials engineer for Joseph K. Knoerle and Associates, Inc., general consultants on the Northern Illinois Toll Highway. Mr. Waddell is chairman of ACI Committee 609, Compaction of Concrete by Mechanical Means, and a member of ACI Committee 611, Inspection of Concrete.

SPECIFICATION REQUIREMENTS

Basically four classes of concrete were specified (Table 1):

Class K—foundations of structures and other massive sections
Class M—general structural purposes
Class P—paving concrete
Class R—prestressed members (over 200 bridges)

Concrete was considered as having failed to meet requirements if more than four consecutive tests in Class P or three consecutive tests in all other classes fail to reach design strength.

To obtain and maintain concrete meeting these requirements was by no means automatic. Care and effort in planning and supervision, and in indoctrination and inspection, were required to increase the probability of acceptable results.

Within the limits of minimum cement content, maximum water-cement ratio, maximum slump, maximum size aggregate, and maintenance of entrained air within allowable limits, no other restriction was placed on how the contractor shall obtain the required percentage of tests above the design strength.

With this type of specification, the more uniform the contractor's operations, the lower the probable average strength needed to meet the requirements that a given percentage of the tests be above the design strength for

TABLE 1—CLASSES OF CONCRETE

Class	28-day design strength, psi	Required percent of tests to exceed design strength	Maximum *W/C*, gal. per sack	Maximum slump, in.	Entrained air, percent	Minimum cement, sacks per cu yd	Maximum aggregate nominal size, in.
K	3000	90	5¾	3	4 ±1	5	2
M	3000	90	5½	3	4 ±1	5¼	1
P	4000	80	5½	1½	4.5±1	5½	2
R	5000	90	5	3	3 ±1	6¾	1

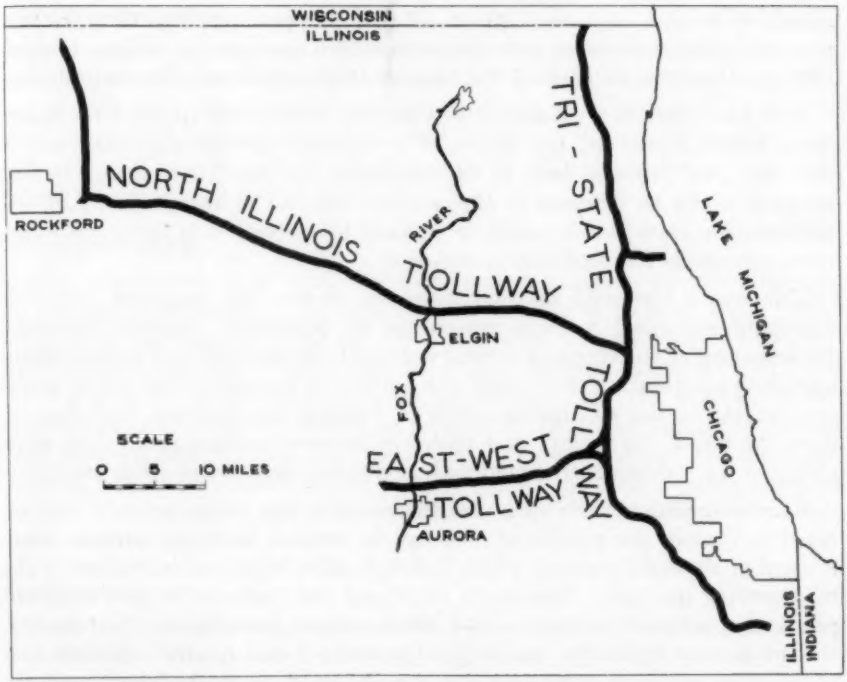

Fig. 1—Northern Illinois Toll Highway

each class. In contrast, lack of uniformity requires a higher average strength to meet the requirements, which automatically penalizes the contractor by the need for a higher cement content. Thus the contractor is motivated to uniform operation, resulting in higher quality work, instead of gearing everything to the poorest quality expected.

To help the contractor maintain as uniform an operation as possible, "self-functioning" and automatic operations and devices were used to the utmost. Basically, the most important of these were automatic batching plants, finish screening operations at the last handling of the aggregate to minimize segregation and to reject fines, electric recording moisture meters that provide a continuous record of the moisture in the sand batches to permit water and aggregate weight adjustments, and automatic recorders that permit a study of operations and indicate variations, thus enabling the engineer to trace causes.

CONTROL OPERATIONS

Aggregate investigations

With the anticipated number of contractors, it was impossible to predict the sources of aggregates. The short time available for preliminary investigations made it impossible to locate deposits, test them thoroughly, and

specify or restrict their use. About all that was practical was to make sure that the quantities needed were available within a reasonable radius of operation, and that from all available information there would be no serious problems.

As a final protective measure it was decided that source approval for aggregates would depend on the results of a thorough petrographic examination and the usual physical tests of the materials. It was felt that any serious problem would be detected in this manner and any warnings raised by the petrographic examination would be checked by inspection of in-service structures containing the materials in question.

This was a fortunate decision inasmuch as the first proposed source of aggregate contained a large percentage of potentially reactive particles. Examination of structures in service indicated the possibility of a mild alkali-aggregate reaction, which in itself was not felt to be serious, but which might open cracks to the weathering action of freezing and thawing, and thus reduce the life of the facility and probably increase maintenance. The same situation was subsequently found with aggregate from several other sources.

Since observations indicated that the reaction was rather mild, it was decided to explore the possibility of obtaining cement with a maximum alkali content of about 0.7 percent, which looked feasible from past operations of the mills serving the area. This was a calculated risk, believed to give sufficient protection, without imposing a low alkali cement restriction. Fortunately, the 0.7 percent limitation was attainable where it was needed. Several contracts were committed before these details could be worked out, and the use of potentially reactive aggregate with the higher alkali cement on these early contracts was permitted as a calculated risk justified only because of the mildness of the observed reaction.

Production control

Once a source of aggregate was approved, inspection of production and certification of shipments was taken over by the commercial testing agency engaged by the toll highway commission. The testing agency maintained inspection at the plant wherever production for the toll highway was in progress. Routine gradation and cleanliness tests were made, and at given intervals samples were sent to the laboratory for soundness, abrasion, specific gravity, and absorption tests. Whenever the character of the product appeared to change, another sample was taken for petrographic examination. By breaking down the inspection into such simple operations, quality control for large tonnage of materials was possible without the need for highly trained inspectors. Only a few experienced field materials engineers were needed to supervise the over-all operations. This was simply the adaptation of industrial quality control to construction operations. Without this system it would have been impossible to maintain the high degree of quality desired, with the fast schedule of operations. Results have been gratifying, and have shown the plan to be practical, workable, and efficient.

Cement investigation

The cement investigation was limited to ascertaining that there would be sufficient material available when needed, and that the adjustment and shifting around necessitated by the aggregate problems were feasible. The cement was adequate in quantity and in performance, and no special problems developed. All cement was reserved in bins for the toll highway, and was tested for compliance before shipment.

Mix proportioning

Mix calculations were made by each section engineer for the contracts he was administering, and for each class of concrete to be used, using "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)." To maintain reasonable uniformity, the field material engineers employed by the general consultant cooperated in this activity. In general, it was possible to keep the sand content at around 30 percent or slightly lower and thus obtain a concrete that has a minimum unit water content and the maximum of the more durable coarse particles. Mix proportions were adjusted in the field for actual operating conditions after the batch or central mix plants started operating.

Field laboratories

Each section engineer maintained a field laboratory in which final gradation and cleanliness checks were made on the aggregate, and where mixes were proportioned. In addition, all checking and control of the actual production of mixed concrete, including preparation of concrete cylinders, yield tests, and air content determinations, were made or supervised by the field laboratory.

Concrete laboratory

The testing agency set up a fully equipped laboratory near the center of the project. This laboratory inspected and tested the aggregate, base, and subbase production; collected, capped, and tested concrete cylinders; and sampled, tested, and certified for shipment all cement that was bin-sealed for the project.

Operations were standardized to increase uniformity; unavoidable variations were treated so as to make their effect constant insofar as possible. The same individual performed the same task from day to day, in the same prescribed manner. A specially fitted truck with padded boxes, each made to carry one pair of cylinders, maintained a regular schedule to pick up cylinders made in the field the previous 48 hr. The cylinders were tagged with special metal tags with noncorrodible wires embedded in the concrete. These were coded and the cylinders were accompanied by a field sheet that gave all necessary information about the concrete from which the cylinders were made. This information appeared in the final cylinder strength reports. Thus there was no possibility of mix-up except for errors made by the inspector in recording facts. When the truck returned to the laboratory, the cylinders were unloaded using an exact routine, and taken into the moist room. The

following morning they were taken out and capped in a prescribed manner, and then returned to the moist room until ready for testing.

At set intervals, a cylinder was broken down completely to observe the type of break and appearance of the concrete. Unusually weak cylinders are also broken down in a like manner for inspection. Reports are distributed to the parties concerned the same day in most instances.

The only variable left is in making cylinders and in handling them on the job until picked up by the central laboratory truck. Utmost uniformity was fostered through indoctrination of field personnel, constant checking, and as close coordination by the general consultant as possible. Notwithstanding all these precautions, there were definite and expected variations of significance attributable to this phase of the operation. The aim was to attain as uniform an operation as practical, without delaying the work for unnecessary refinements.

Sampling of concrete

For effective and economical sampling and testing, small samples must be taken at frequent and regular intervals. To make such sampling useful requires analyzing the variations to determine whether they are of statistical significance, or are only chance variations within the quality range established for the job.

For the concrete on this project, samples consisting of two cylinders each were taken at frequent intervals in accordance with a predetermined schedule as follows:

1. Three pairs of cylinders, each pair taken from a separate batch, for each 2000 sq yd of pavement, or fraction of this quantity placed each day.
2. Three pairs of cylinders, each pair taken from a separate batch, for each 100 cu yd of structural concrete or fraction of this quantity placed each day.

Two cylinders were taken for each set, instead of the usual three, so that for the same expenditure of energy and cost (six cylinders total), 50 percent more batches were sampled, thus adding statistically significant data without added expense. The plan was to test one cylinder of the first pair at 7 days and one at 28 days, one cylinder of the second pair at 7 days and one at 28 days, and both cylinders of the third pair at 28 days. The 7-day tests properly interpreted, indicated the probable expected 28-day strength. This permited adjustment of operations without waiting for the 28-day breaks in many cases. The third pair provided the range in strengths to evaluate uniformity in handling and making of the cylinders. In general, this program was based on the fundamental principles outlined in "Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)."

Batching and mixing plants

To attain the desired batch-to-batch uniformity, semiautomatic plants provided with electric moisture meters, recording charts, finish screening of aggregates immediately before discharging into the plant hoppers, and interlocking gates, were specified for paving work, and for structural work in-

volving large volumes of concrete. The automatic feature is needed anyway in paving work to keep up with two dual drum pavers. The recording feature does not add much to the cost of the plants and provides a permanent record that can be studied as problems arise. Requiring automatic plants for the larger volume of structural work has made it advantageous to set up central concrete plants to serve several contracts from one plant. This has also reduced inspection and improved quality through fewer operations. On the Tri-State Tollway, four plants satisfactorily supply structural concrete for 82 miles of roadway.

EXAMPLES OF CONTROL

Some examples of how this type of control operated during the first season will now be described.

Control charts were kept for each contract and for each class of concrete in accordance with the "Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)." A typical chart is shown in Fig. 2. Both the 7-day and 28-day curves are plotted to permit an evaluation of the probable 28-day strengths from the 7-day results. This is necessitated by the speed of the work and the need of making decisions long before sufficient 28-day strengths are available.

The top half of the chart has the individual compressive strengths of sets of cylinders for both 7 days (shown by dotted lines) 28 days (shown by full lines). These curves show how many sets are below the specified strength, and therefore whether or not specification requirements were being met. From the 7-day strengths it was possible, after experience with a given mix, to determine whether the 28-day strengths are likely to meet specifications.

The bottom half of the chart shows similarly the moving 5-set averages of the 7-day and 28-day strengths. For forecasting from the 7-day strengths, and for evaluating trends, this is more valuable than the curves in the top half of the chart. Operations were evaluated for each 30 sets of cylinders and adjusted accordingly, except where the 7-day strengths indicate the desirability of adjustments before results from 30 sets become available.

Fig. 2 shows an actual operation on one of the contracts. This operation started under proper control from the beginning, in contrast to many others that required a period of adjustment. The chart represents the operation on one contract for Class M structural concrete. While the coefficient of variation of 14.4 percent for the first 30 sets of cylinders is only average, 100 percent of the strengths were above the 3000 psi specified for this class. This is in excess of the 90 percent required for compliance. By the time the first 40 tests on the 7-day breaks were available, it was evident that the moving 7-day average had stabilized around the 3000 psi level, and therefore a reduction in cement would be justified. Although only twenty 28-day strength tests were available it was not deemed necessary to wait for the 30-set evaluation. A reduction from a cement content of 5¾ sacks per cu yd to 5¼ was

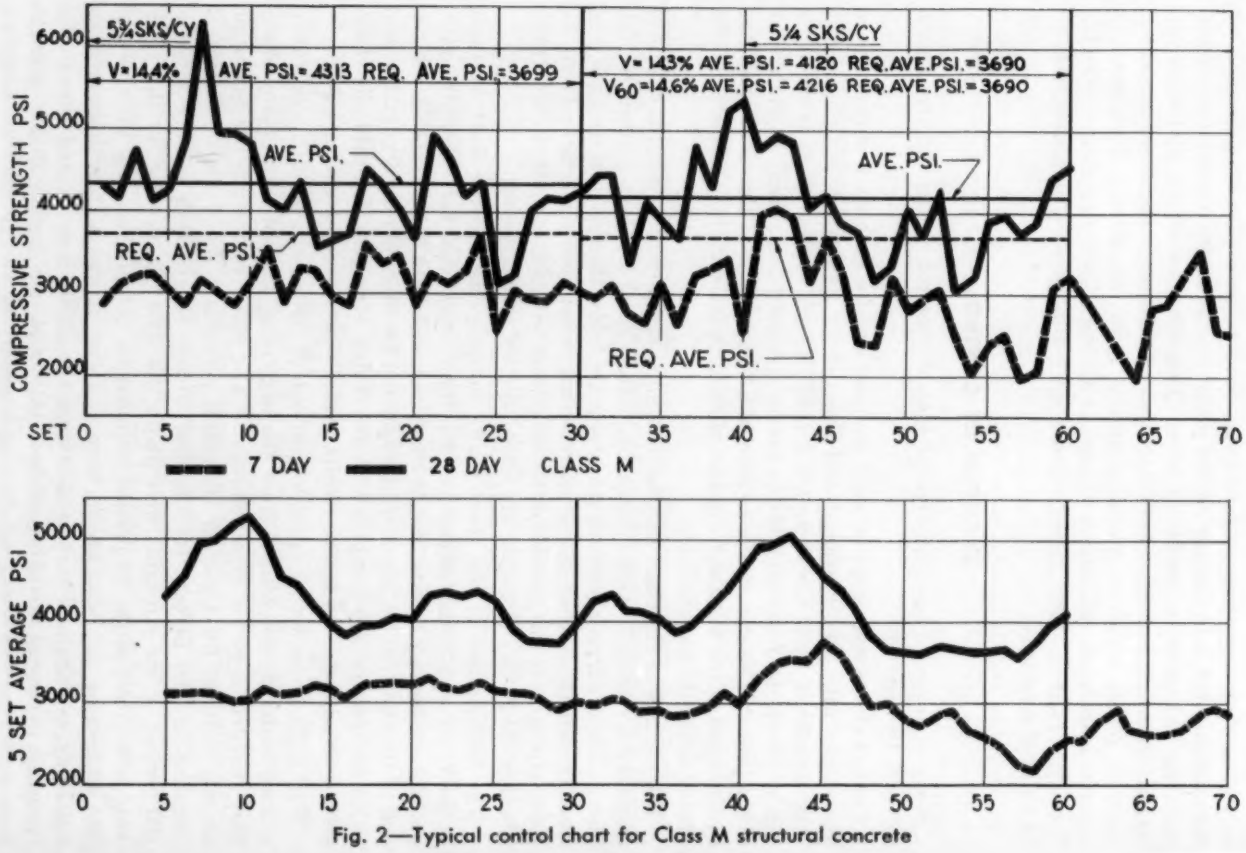

Fig. 2—Typical control chart for Class M structural concrete

therefore allowed. The chart shows that the strengths dropped, but still met the specification requirements that 90 percent of the tests equal or exceed 3000 psi. Further cement reduction was not possible under the specifications as 5¼ sacks per cu yd was the minimum allowed for this class of concrete.

On the same contract and under the same supervision as the operation of Fig. 2 was some Class K structural concrete used for foundations and massive sections (Fig. 3). This class had a larger maximum size aggregate, lower cement content, and two sizes of coarse aggregate as against one size for Class M. On most contracts, the volume of Class K concrete was small. After

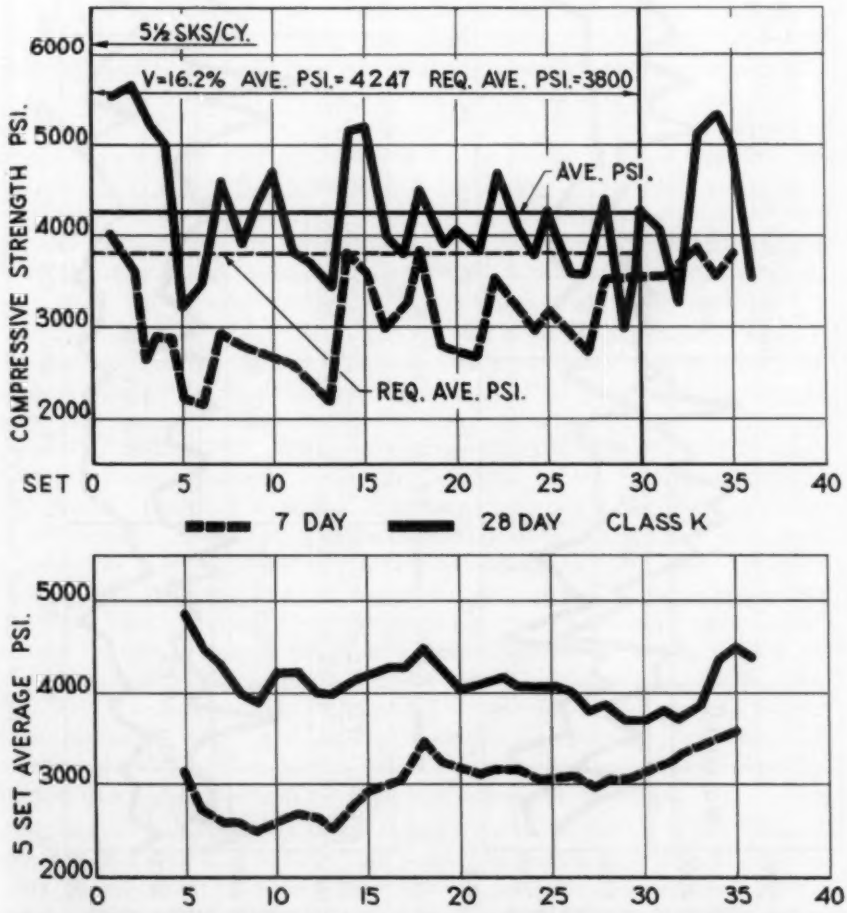

Fig. 3—Control chart for Class K structural concrete showing satisfactory control from the beginning

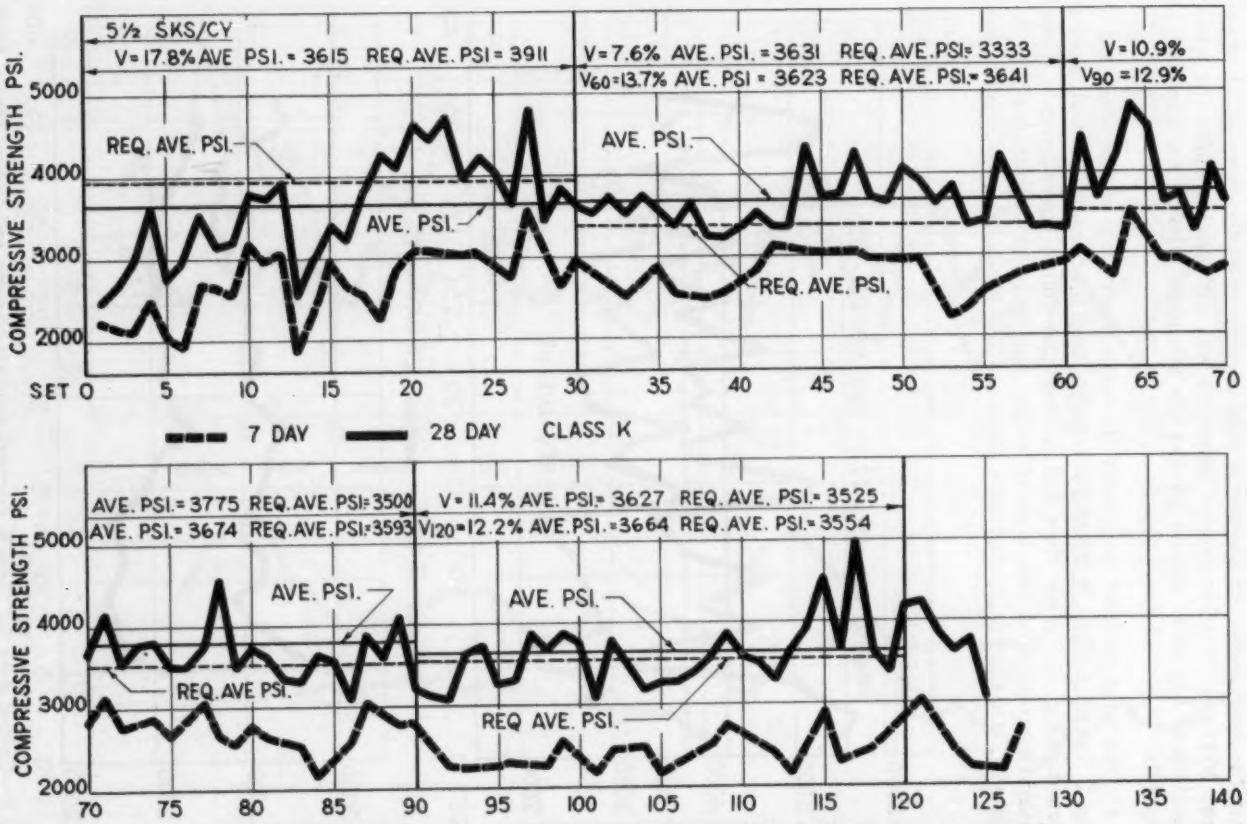

Fig. 4—Control chart for Class K structural concrete showing improvement in control

trying it for a short period, this contractor decided to use Class M instead and avoided batching two classes of concrete. Control was satisfactory from the beginning with a coefficient of variation of 16.2 percent, not much different from the 14.4 percent obtained on Class M. Part of the difference can be accounted for by the variation introduced by the larger aggregate particles in the confines of the 6-in. diameter cylinder specimens.

A third type of operation was one which started with the usual difficulties and then straightened out. In this case (Fig. 4), involving Class K concrete, many of the low strengths at the start were probably due to the larger size aggregate particles getting together in the cylinder and forming a honeycomb pocket. Such pockets were noticed in many of the broken low strength Class K cylinders. The inspectors were instructed to pick the 2-in. particles out as they were making the cylinders, but it took a little time for them to develop the skill to do the job effectively. From a rather high coefficient of variation of 17.8 percent for the first 30 sets, a remarkable improvement to a coefficient of 7.6 percent occurred in the second 30 sets, but finally settled to the real level of operation of 10.9 percent and 11.4 percent in the successive groups of 30's. The over-all coefficient for the whole operation was 12.2 percent, which is better than average. A small drop in strength would most likely have resulted in more than three sets of cylinder strengths below 3000 psi, and therefore outside the specifications. No decrease in cement content could therefore be allowed. An improvement in uniformity would probably have permitted such a reduction, with resulting savings to the contractor.

Still another type of operation is shown in Fig. 5. This represents Class R concrete, used for prestressed members, with a requirement of 5000-psi compressive strength at 28 days, and a minimum of 90 percent of the tests to equal or exceed this level. After the usual starting difficulties, two things may be noticed: (1) an increase in average strength from 5171 psi to about 5900 psi, where it hovered since; (2) a decrease in the coefficient of variation from 13.1 percent to around 7 percent, with a good over-all coefficient of 10.7 percent. Those two changes indicate definite improvement in control. A reduction in cement could easily have been made and still have met specification requirements. But the contractor elected not to do so because he wanted to take advantage of the faster gain in strength provided by the higher cement content to permit earlier stripping of the forms and moving of the beams off casting beds to increase production. In fact, after the 65th set of cylinders, a shift was made to Type III cement to speed up production. It might be worth thinking about the desirability of putting into prestressed concrete specifications some provision to permit the engineer to force a reduction in cement if he finds it to the best interest of the owner.

Fig. 6 shows an operation that was bad from the beginning. The strengths were low and remained so for a long period. It took a long time to convince everyone concerned that the cement content needed to be increased. Every possible other corrective measure was tried before going to a higher cement content at the 90th set of cylinders. The cement should have been increased

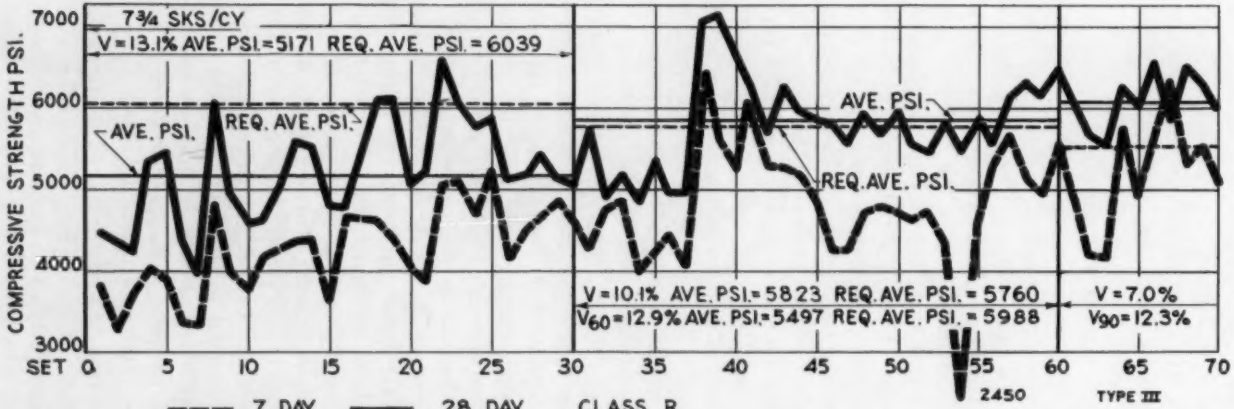
7¾ SKS/CY
V = 13.1% AVE. PSI. = 5171 REQ. AVE. PSI. = 6039
AVE. PSI.
REQ. AVE. PSI.
V = 10.1% AVE. PSI. = 5823 REQ. AVE. PSI. = 5760
V60 = 12.9% AVE. PSI. = 5497 REQ. AVE. PSI. = 5988
V = 7.0%
V90 = 12.3%
COMPRESSIVE STRENGTH PSI.
SET 0
2450
TYPE III
7 DAY
28 DAY
CLASS R
AVE. PSI. = 6084 REQ. AVE. PSI. = 5505
AVE. PSI. = 5693 REQ. AVE. PSI. = 5938
V = 7.2% AVE. PSI. = 6023 REQ. AVE. PSI. = 5519
V120 = 8.7% AVE. PSI. = 5775 REQ. AVE. PSI. = 5624
V = 7.1% AVE. PSI. = 5964 REQ. AVE. PSI. = 5325
V150 = 10.7% AVE. PSI. = 5813
REQ. AVE. PSI. = 5500

Fig. 5—Typical control chart for Class R concrete

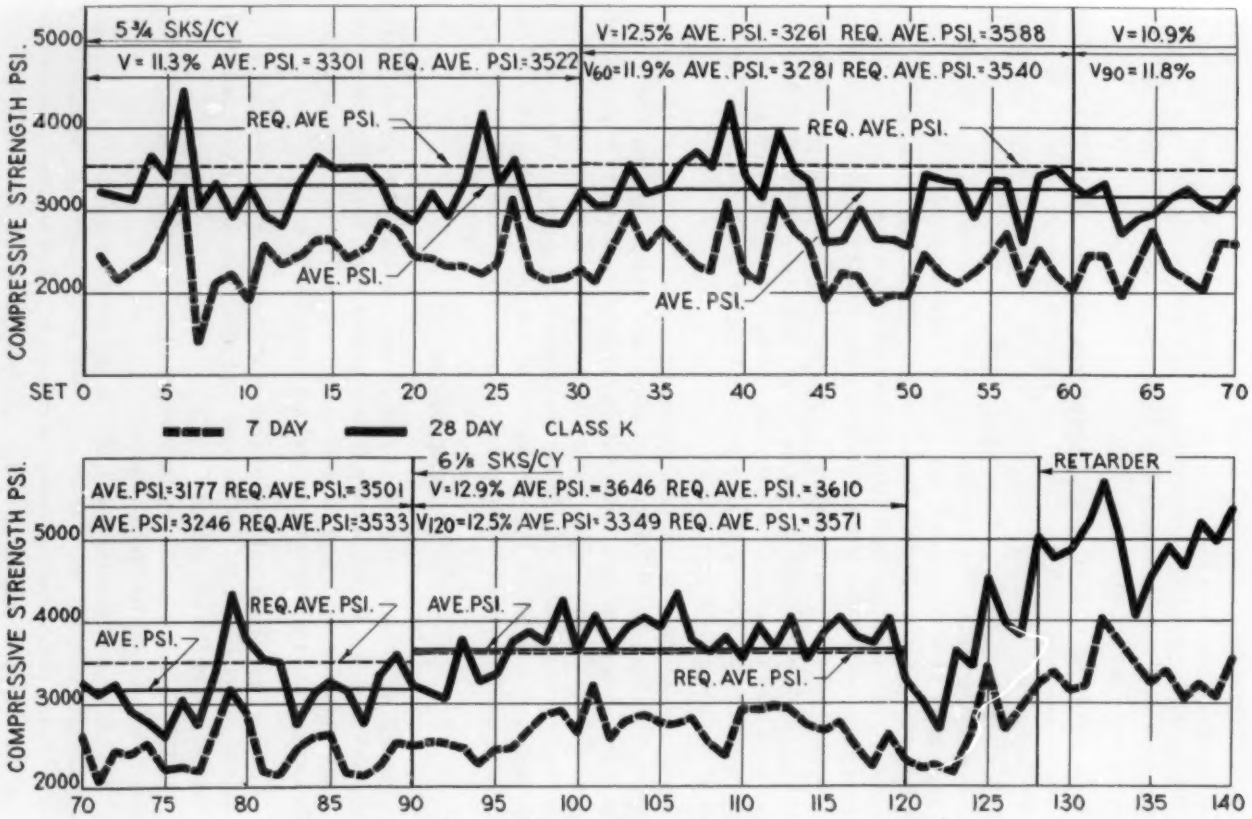

Fig. 6—Control chart for Class K concrete illustrating poor control

soon after results of the first 30 sets became available. Strangely enough, the coefficient of variation remained about the same irrespective of cement content, ranging from 11.3 percent to 12.5 percent. The engineers became so conscious, however, of the importance of the cement content that they experimented with water reducers, and the results of these tests show the short upward trend toward the end.

It is gratifying that of some 50 contracts, most of which involve three classes of concrete, each starting in a different manner, with 23 different firms doing the engineering supervision, the end of the first season saw practically all of them meeting the strength, water-cement ratio, slump, and air content requirements. Yet six different sources of cement and over 25 sources of aggregates were being used, introducing large potential variations. The advantage of reduction in cement was not used as much as it might have been because by the time the various operations were brought under control cool weather had arrived and it was feared that reduction in cement might bring about a great deal of trouble and unbalance the control that had been attained. Some of the better controlled contracts did, however, succeed in taking advantage of such reductions. A few of the contractors were so close to finishing operations for the season that they preferred not to change, with the possible risk of upsetting their performance.

Additional information can be obtained from a careful study of the control charts. For example, differences in inspection and in making and taking care of cylinders in the field become apparent from these charts when concrete originates from the same plant, but is supervised by different groups. All the possible information that can be derived from the control charts is not yet apparent, but it is hoped that as additional data become available and are analyzed, much more can be learned about this phase of the work.

CONCLUSIONS

Although no definite conclusions and recommendations are justified from this short analysis of preliminary data obtained from beginning operations of many contracts, all trying to score some progress before cold weather, the following tentative statements are perhaps warranted:

1. This type of control is workable as evidenced by the practically 100 percent compliance with the strength and other concrete requirements before the end of the first season.

2. This type of control is realistic in providing tolerances that are reasonable of attainment by the contractor.

3. This type of control is effective in reducing cement to the minimum required to meet the specifications for any given operation, improves uniformity, and provides a tool to set up standards and to measure attainments.

4. This type of control provides inducements to save cement. To take advantage of such saving, the contractor has to improve operating efficiency,

and by doing so finds it possible to reduce costs and to cut the time required to complete the work.

It is felt that this type of control is a definite advance over a static minimum strength requirement and improves the quality of the work without being burdensome.

ACKNOWLEDGMENTS

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Title No. 55-62

Role of CEMENT in the CREEP of MORTAR

By A. M. NEVILLE

CREEP OF MORTAR SPECIMENS made with 15 different portland cements and subjected to a sustained compressive stress was measured over a period of several months. These tests, involving as many as 700 specimens, indicate that creep is approximately proportional to the ratio of applied stress to strength of mortar at the time of load application, regardless of the identity of the cement. The relationship between creep and the stress-strength ratio appears to hold good for both dry and humid storage conditions, provided specimens are free from shrinkage. A wide range of stresses has been investigated and, for the mix proportions used, the stress-strength ratio seems to be the most important factor influencing the magnitude of creep. Cement fineness was not found to affect creep in the range tested.

The paper reviews in some detail factors influencing strength of cement, and also discusses the creep data of other investigators in relation to the proposed creep versus stress-strength ratio relationship.

CREEP IS AN INHERENT PROPERTY of concrete and no concrete structure subjected to a sustained load is free from the effects of creep; yet it is probably the least well known property of concrete. The mechanism of creep has been discussed by the author in an earlier paper,[1] and the effects of creep in various types of structures and its importance in structural design have also been described by him[2] and need no further emphasis.

Most attempts during the last 30 years to predict the magnitude of creep for any type of concrete were in the form of empirical relations between creep and some variable in the concrete mix proportions. This approach, probably due to the scarce theoretical background on creep, was unlikely to produce more than a fragmentary picture since it is generally difficult to alter only one variable in the properties of a concrete mix; thus the true cause of creep variations may often have been obscured.

Although the properties of the aggregate affect the creep of concrete as a whole, there is little doubt that it is the hydrated cement paste that is primarily responsibie for creep.[1] For this reason it was thought that an investigation into the effect of the properties of cement on creep should prove useful; indeed, such a relation might be revealing regarding the mechanism of creep.

Existing information about the role of cement in creep is scarce, and is particularly difficult to evaluate since it is mostly qualitative as far as the properties of cement are concerned. Cements have often been described only by their type, and, as will be shown later, the differences in creep of cements of the same type can be considerably greater than between cements of nominally different types.

A. M. NEVILLE, lecturer in engineering, University of Manchester, England, has conducted considerable research on creep since 1950 and published several papers on the subject. Mr. Neville has also been lecturer in engineering at Canterbury University College, Christchurch, New Zealand, and the University of Southampton, England.

The results of the tests published to date have been recently reviewed by the author.[3] The general conclusion which can be drawn is that creep seems to be inversely proportional to the rapidity of hardening of the cement used. Thus at a given age, creep would be in an increasing order for concretes made with the following cements: aluminous; rapid-hardening (Type III); standard portland (Type I); portland blast furnace; low heat (Type IV); and portland pozzolan. The order of the last three is not clearly established since the test conditions were not directly comparable.[4,5] It is well known that at the same age these cements have achieved varying proportions of their final strengths, and it seems logical that the more hardened the paste the more rigid it is and the lower its creep potential. Effects of the absolute value of the paste strength will be described later for portland cements.

The present paper gives data on the creep of mortars made with 15 different portland cements and stored under carefully controlled conditions of temperature and humidity, and shows that the inherent strength of cement is a primary factor influencing creep. If this relation between creep and strength means that they are both affected by the same properties of the hydrated cement paste, then this should throw some light on the mechanism of creep and help in understanding its nature.

INFLUENCE OF THE COMPOSITION OF CEMENT ON COMPRESSIVE STRENGTH

The influence of the chemical composition of cement on compressive strength of mortar and concrete is of particular interest since, as will be shown later, creep and strength are closely related.

One of the earlier observations relating to the strength development of cement concerned the rates of hydration of C_3S and C_2S—two silicates primarily responsible for the strength of hydrated portland cement.[6] A convenient, though approximate, rule assumes that C_3S hydrates completely in the first 4 weeks while C_2S does not start its process of hydration until 4 weeks after the gaging with water;[7] at about 1 year the two compounds, weight for weight, contribute approximately equally to ultimate strength.[8] Testing neat C_3S and neat C_2S, Brown and Carlson[6] found the strength of each at 18 months to be 10,000 psi; at 7 days C_2S had no strength while the strength of C_3S was 6000 psi.

Influence of the other major compounds of cement on strength development has been established less clearly. Gonnerman[9] observed that C_3A contributes to the early strength of cement paste but causes retrogression later, particularly

in cements with a high C_3A or ($C_3A + C_4AF$) content. Bogue and Lerch[10] considered C_3A not to be a true cementitious material, and suggested that the influence of C_3A was only a secondary effect in that the rapid hydration of C_3A decreases the "effective" water available for hydration of the silicates; this may account for the apparent early positive contribution of C_3A. C_4AF was thought by Gonnerman[9] to be of little influence on strength. Davis[8] found the contribution of C_4AF to be comparatively small but consistently negative. According to Lea[11], C_4AF hydrates rapidly and develops some strength. Bogue and Lerch,[10] however, confirmed the retrogression effects of C_4AF and ascribed them to the colloidal hydrated $CaO \cdot Fe_2O_3$ deposited on the grains of the other compounds, thus delaying their reaction with water.

Davis and co-workers,[8] and later Gonnerman,[9] tested many cements and on the basis of their results suggested an empirical formula for prediction of strength of cement paste at various ages, of the type:

$$\text{strength} = a \cdot (C_3S) + b \cdot (C_2S) + c \cdot (C_3A) + d \cdot (C_4AF)$$

where the chemical symbols represent the percentage by weight of each compound, and a, b, etc. are constant parameters representing the contribution of 1 percent of the corresponding compound to the paste strength.

The attractiveness of such a formula is immediately apparent and the prospect of easy strength forecast at the cement plant, instead of the tedious and slow briquet and cube tests, is appealing. However, while the influence of the silicates can be predicted reliably, unfortunately effects of the other compounds, as mentioned earlier, are not clearly established. According to Lea[12] these discrepancies may be due to glass formation in the clinker. Alumina and ferric oxide are completely liquefied at clinkering temperatures, and on cooling, crystallize into C_3A and C_4AF. The extent of glass formation would thus affect these compounds to a large degree while the silicates, which are formed mainly as solids, would be relatively unaffected. The glass may also hold a large proportion of "impurities" such as the alkalies[9] and MgO;[13] the latter is thus not available for expansive hydration. It should be remembered that the Bogue compound composition assumes that the clinker has crystallized completely to yield its equilibrium products. It is well known that the rate of cooling of clinker affects the degree of crystallization with a resulting varying amount of glass. The reactivity of glass is different from that of crystals of similar composition, and hence normally similar cements may show different strength development depending on the glass content.

An interesting effect of the glass content was found in connection with some tests on the resistance of various cements to sulfate attack;[11] contrary to expectations, some cements with a high C_3A content showed a high resistance. This was because C_3A failed to crystallize during the cooling of the clinker and was predominantly present in the glass; thus the active C_3A content was considerably lower than that calculated by the Bogue method. The glass content is also of importance as far as the process of the manufacture of cement is concerned since glass seriously affects the grindability of clinker.[14]

It seems, therefore, that the rate of cooling of clinker, as well as possibly other characteristics of the process of cement manufacture, affect the strength, and defy attempts to develop a formula of the type suggested by Gonnerman. Nonetheless, if one process is used and the rate of cooling is kept constant there seems to be a definite relationship between strength and compound composition. Indeed, this has been proved to exist by tests, as yet unpublished, at one particular plant.

This possibility of additive properties of the compounds is of considerable interest since the structure of the set cement mass as a whole might be reasonably expected to depend on the conditions of setting under which it is first established, and not only on the relative amounts and the specific individual properties of the compounds.[10,12]

As early as 1934 Eiger[15] observed that a graph of the degree of hydration versus strength plots one curve for a variety of cements, and he checked this relationship both in compression and in tension.[16] Powers and Brownyard[17] extended this conception in their gel-space ratio hypothesis, according to which strength of cement is proportional to the cube of the ratio of the volume of the cement gel to the original space available, regardless of age, water-cement ratio, or identity of cement. They observed that the total surface area of the solid phase is related to the computed cement composition, and this composition affects the value of the ultimate strength. Powers[18] later thought, and here he is at variance with Eiger, that all compounds in the cement hydrate at the same fractional rate; this suggestion is highly controversial.

Effects of the minor compounds have not been reliably established; they are generally thought unimportant as far as strength is concerned, although K_2O is believed to replace one molecule of CaO in C_2S with a consequent rise in C_3S content above that calculated.[19] Gypsum is unimportant provided the cement is properly retarded. Fineness affects the rate of gain of strength but generally not its ultimate value.

From test data reviewed above it is impossible to state beyond any doubt a definite relationship between the characteristics of cement and its strength, nor is it possible to say with certainty what is the physical mechanism of strength of cement paste. It seems, however, that the physical properties of cement gel and its volume relative to the space available are the two vital factors. It will be shown later that creep of mortar is directly related to its strength, i.e., whatever the factors are which affect strength—be they compound composition of cement or degree of hydration—the same factors affect creep: the weaker the cement paste the greater is its creep capacity under a given load. Alternatively, a paste loaded to the same proportion of its strength exhibits approximately the same creep, i.e., creep would appear to be a function of the *relative* amount of unfilled gel space, and it is possible that it is the voids in the gel which are responsible for both the relatively lower strength and higher creep.

EXPERIMENTAL DETAILS

To test a large number of mortar specimens subjected to sustained loading it was necessary to devise a loading frame in which the load could be easily measured and kept constant. Hydraulic loading was considered too expensive and for that reason a lever-spring frame, working on the principle of a nutcracker was designed (Fig. 1). The frame has already been described in some detail.[20]

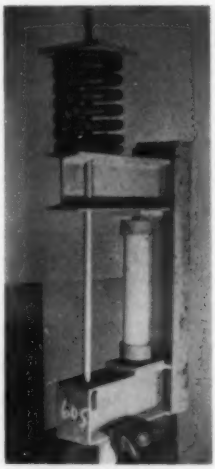

Fig. 1—A loading frame

Specimens

For the basic series of tests, mortar of fixed mix proportions was used; it had a water-cement ratio of 0.4, aggregate-cement ratio of 3.0, and Leighton Buzzard standard sand (similar to Ottawa sand) as aggregate. Thus quality (composition) of cement was the only variable, and, furthermore, the use of pure silica one-size aggregate meant that it was least likely to affect creep except in so far as the presence of aggregate represents an inert body dispersed in the cement paste.

The specimens were 2 in. diameter cylinders, 9¼ in. long, their somewhat unusual height-diameter ratio being governed by the minimum length necessary for measurement of creep on the one hand, and by the maximum diameter permissible for the stress desired, on the other. Since the object was to measure variation in creep rather than its absolute value, the shape of the cylinder was not thought to be important.

Loading and curing conditions

There were 24 loading frames. All specimens were loaded at 28 days and kept in one of four storage cabinets. Each cabinet was thermostatically controlled at 20 ± 0.1 C, but the temperature of the room in which readings were taken varied sometimes by as much as ± 2 C. Humidity in the cabinets was kept constant by means of a bath containing a salt solution with the solid phase present, a fan aiding uniform distribution of humidity within the cabinet. $CaCl_2 \cdot 6H_2O$ gave a relative humidity of 32 percent (referred to as "dry") while the relative humidity of 95 percent (called "humid conditions") was obtained by the use of water. Both temperature and humidity were continuously recorded. Wet and dry bulb thermometers graduated to 0.1 C and placed in the vicinity of the fan were used for control purposes in the humid cabinets.

All specimens were cured from the age of 24 hr (when stripped) at the same humidity as well as temperature as that at which they were subsequently to be stored under load. The method of curing all specimens under the same conditions as those under which they were subsequently stored under load is believed to have produced specimens nearly free from shrinkage, and thus the change in strain with time under load could be assumed to represent creep only. It may be relevant to note that in the past the majority of investigators have used unloaded control specimens whose shrinkage was measured; shrinkage of the loaded specimens was believed to be the same, and thus the total movement of loaded specimens adjusted by the shrinkage of unloaded companions was assumed to represent "true" creep An objection to this method is that, as far as we know, a creep-shrinkage interaction is not impossible. For instance, according to Pickett,[21] during drying shrinkage gel particles change their relative positions, some making closer contacts while others separate. This distortion is random in direction and the amounts of "make" and "break" are equal. When, however, a directional stress is added to the process of drying, a large resultant distortion occurs. This represents the increase in creep over that in a nondrying specimen. After the gel particles have acquired stable positions the rate of creep decreases considerably.

As will be seen later, for the two conditions of humidity used in the present series of experiments (with practically no shrinkage taking place) creep was found to be independent of humidity conditions.

Control specimens

As a check, conventional companion specimens were used for the majority of the loaded specimens, and, indeed, some shrinkage was measured. This varied between 2×10^{-5} and 8×10^{-5} for both the humid and the dry specimens, with the exception of Cement 2 when dry. The behavior of this cement under dry conditions was found to be in many respects anomalous and will be discussed more fully later.

Cements used

Fifteen portland cements were used; No. 1 to 8 were usual commercial cements manufactured in England; No. 9 to 11 were specially produced in the United States.[22] In addition, two of the cements were reground to two additional finenesses each, and also some tests were made on an aluminous cement (No. 12) for the purpose of a general comparison with the portland cements. Full data on the cements used are given in Table 1, from which it can be seen that the range used included not only ordinary and rapid hardening cements but also low-heat, sulfate resisting, and white cements. Table 2 gives the characteristics of the aluminous cement (No. 12).

CREEP OF MORTARS MADE WITH VARIOUS CEMENTS AND SUBJECTED TO THE SAME STRESS

From measurements of the variation in strain with time in the specimens subjected to a sustained load a family of creep-time curves was obtained for the various cements. All specimens stored under dry conditions were subjected to a stress of 1430 psi. For the humid condition two sets of specimens were tested, one stressed to 1430 psi, the other to 2150 psi. Tables 3 and 4 give a summary of the creep data together with values of the compressive strength of companion specimens.

For each test two to eight similar specimens were used to obtain reliable values. Needless to say, observed strains were not identical in all specimens owing to the inherent lack of homogeneity of a mortar mix and to the unavoidable mix-to-mix variation. In most cases good agreement within each test was obtained, although for some cements the variation in strain was $\pm 5 \times 10^{-5}$ from the mean. This is rather smaller than the variation in strength within groups of similar specimens.

As mentioned earlier, the shrinkage of companion specimens was always less than 8×10^{-5}, and the creep values were therefore not corrected for shrinkage. Cement 2 under dry storage conditions proved an exception, and exhibited shrinkage of 13×10^{-5} at 30 days after loading and 23×10^{-5} 30 days later. For the sake of consistency the creep values for this cement were also not corrected for shrinkage.

Creep values given in Tables 3 and 4 show that even within the group of the usual portland cements there are large differences in creep. For instance, creep strains of Cements 1 and 7, stored under humid conditions at a stress of 2150 psi for 120 days are 73×10^{-5} and 44×10^{-5}, respectively, and yet

TABLE 1—PROPERTIES OF THE PORTLAND CEMENTS TESTED

Cement No.	Bogue compound composition, percent				Alkali content, percent		Gypsum content as percent of SO_3	Specific surface, sq cm per g	Nominal description
	C_3S	C_2S	C_3A	C_4AF	Na_2O	K_2O			
1	42.2	25.3	15.6	6.9	0.18	0.87	2.13	3340	Ordinary
2	23.1	40.0	15.3	9.1	0.23	1.37	2.85	3810	Ordinary
3	37.2	29.6	14.9	7.1	0.18	0.92	2.53	3870	Ordinary
4	37.3	31.2	3.5	16.6	0.27	0.57	3.08	4090	Sulfate resisting
5	36.6	25.4	14.6	11.1	0.21	0.87	2.64	3810	Ordinary
6	32.6	36.0	13.8	6.0	0.25	0.63	2.38	4050	Ordinary
7	35.0	36.5	12.5	5.4	0.16	0.60	2.24	3430	Ordinary
8	50.1	26.5	10.7	1.1	0.15	0.14	2.65	3830	White
9	64.5	10.0	12.1	7.5	0.08	0.23	1.88	3230	Rapid hardening
10	41.0	39.0	3.7	10.0	0.33	0.01	1.88	3630	Low heat
11	29.0	52.0	5.3	9.3	0.08	0.22	1.41	3480	Sulfate resisting
2A	Same as cement No. 2							4090	Reground
2B	Same as cement No. 2							4670	Reground
7A	Same as cement No. 7							3760	Reground
7B	Same as cement No. 7							4250	Reground

these cements are both nominally classified as Type I cements and both were manufactured in England.

At the beginning of this investigation it was thought that the differences in compound composition of the various cements would be clearly reflected in their creep behavior. Consequently, a multilinear regression analysis was made, but unfortunately, no such relationship between creep and the compound composition can be reliably postulated on the basis of results obtained for the 11 portland cements tested. This does not mean that compound composition has no effect on creep but it is suspected that there are other factors which modify the direct influence of chemical composition. The 11 cements used were manufactured at different plants from a variety of different raw materials: chalk and limestone; clay, shale, and marl. They also have unknown temperature histories; in particular, they are likely to differ in the firing temperature and in the rate of cooling, and also in a variety of details such as amount of free lime, degree of carbonation, period of storage, and amount of surface hydration. The latter effect and other effects of exposure may be quite important in slowing down the rate of reaction, since it is known that undue exposure of cement may markedly reduce its early strength. The amount of free lime, as determined, is affected by carbonation due to exposure. Another effect of exposure of cement is the raising of its apparent specific surface since the products of hydration have a considerably higher surface area than unhydrated cement. The temperature history, with the consequent variable glass content of clinker, is believed to be the paramount

TABLE 2—PROPERTIES OF THE ALUMINOUS CEMENT TESTED

Cement No.	Oxide composition (percent)								Specific surface per sq cm per g
	SiO_2	Al_2O_3	CaO	Fe_2O_3	FeO	TiO_2	MgO	R	
12	4.65	38.84	38.35	10.03	5.03	1.95	0.98	1.01	3040

TABLE 3—CREEP AND STRENGTH OF MORTARS STORED UNDER HUMID CONDITIONS

Cement No.	Stress of 1430 psi			Stress of 2150 psi				Compressive strength, psi		Ratio $\frac{S_{148}}{S_{28}}$	Ratio $\frac{C_{120}}{C_{20}}$
	Stress as a percentage of 28-day strength	Creep (10^{-6}) after a period under load*		Stress as a percentage of 28-day strength	Creep (10^{-6}) after a period under load*			at 28 days S_{28}	at 148 days S_{148}		
		20 days	60 days		20 days, C_{20}	60 days	120 days, C_{120}				
1	35	25	40	52½	52	64	73	4080	5870	1.44	1.40
2	38	30	44	57	50	65	73	3770	4990	1.32	1.46
3	29½	27	33	44½	35	44	49	4810	5900	1.23	1.40
4	27	22	30	40½	37	47	50	5280	6900	1.31	1.35
5	37½	22	29	56½	49	64	68	3810	5450	1.43	1.39
6	32	24	35	48	45	59	65	4470	5670	1.27	1.44
7	27	21	30	40	32	41	44	5350	6680	1.25	1.37
8	31½	22	35	47½	47	54	58	4510	6100	1.35	1.23
9				35½	32	44	47	6030	6050	1.00	1.47
10				64	48	55	57	3370	5980	1.78	1.19
11				44	42	47	58	4910	6620	1.35	1.14
12				29½	17	26	31	7270	7300	1.00	1.82

*Load applied at 28 days.

factor affecting cement properties. The effects of glass have been mentioned earlier.

As an indirect check on the role of these unknowns a strength-compound composition analysis of the type used by Gonnerman[9] was made. The contribution of C_3S was found to be highly significant, that of C_2S and alkalies significant, while C_3A and C_4AF were found not to contribute significantly (in the statistical sense) to the strength of mortar. For the mortars tested the strength in psi at 28 days was found to be:

$$76.3\,(C_3S) + 39.6\,(C_2S) - 583.5\,(\text{alkalies})$$

where the symbols in brackets denote percentage of compound present, and the coefficients represent the contribution to strength of 1 percent of the appropriate compound.

This equation cannot be compared with those of Gonnerman[9] since he disregarded the alkalies. Their influence on strength has not been previously

TABLE 4—CREEP AND STRENGTH OF MORTARS STORED UNDER DRY CONDITIONS

Cement No.	Stress of 1430 psi as a percentage of 28 day strength	Creep (10^{-6}) at a stress of 1430 psi after a period under load*			Compressive strength, psi		Ratio $\frac{S_{108}}{S_{28}}$	Ratio $\frac{C_{80}}{C_{20}}$
		20 days, C_{20}	60 days	80 days, C_{80}	28 days, S_{28}	108 days, S_{108}		
1	48	57	84	90	2960	3090	1.04	1.58
2	53	113	152	162	2690	3100	1.15	1.43
3	42	45	67	71	3400	3570	1.05	1.58
4	55½	55	78	85	2570	2640	1.03	1.54
5	52	60	81	85	2730	3110	1.14	1.42
6	47	61	84	87	3040	3220	1.06	1.43
7	64	54	76	79	2220	2300	1.04	1.46
8	88	88	133	148	1620	1750	1.08	1.68
9	59	32	39	40	2410	3430	1.42	1.25
10	95	74	115	129	1500	2100	1.40	1.74
11	82½	60	85	94	1730	2360	1.36	1.57

*Load applied at 28 days.

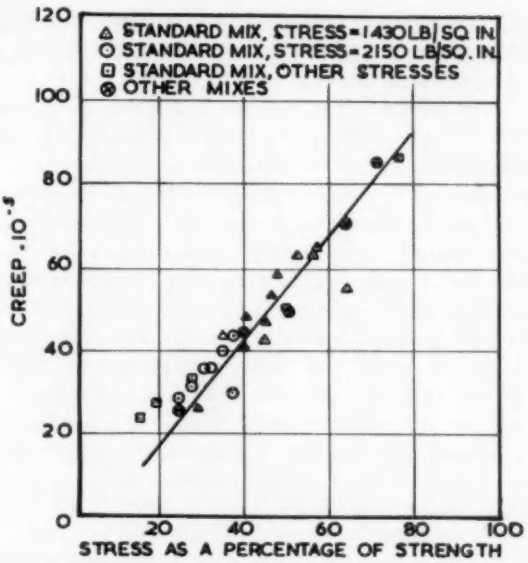

Fig. 2—Relation between the creep of mortars stored under humid conditions for 60 days and the stress-strength ratio (regression line for stresses of 1430 and 2150 psi). See Table 3

observed, and is believed to be of interest. The high and low strength trends of the present equation and of those of Gonnerman are in agreement.

A correlation of creep with the reciprocal of the strength at the time of application of the load was then made, and this showed a high level of significance, in most cases of the order of 0.1 percent. For humid conditions the relation between creep and strength for a given applied stress is unmistakable; a typical graph is shown in Fig. 2. For dry conditions the trend only is apparent; it is not clear whether this is due to an unreliable determination of strength. If, however, data for the dry condition are superimposed on those for the humid condition, as shown in Fig. 3, they all seem to correspond to one linear

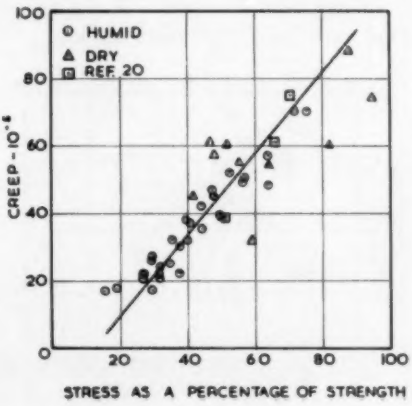

Fig. 3—Relation between creep of mortars stored under load for 20 days and the stress-strength ratio (regression line for all points is identical with the line for humid conditions). See Tables 3-6

relation between the creep and the ratio of the applied stress to the strength, the scatter of results being of the order encountered in most determinations of concrete properties.

It is realized that if creep is a function of strength then, strictly speaking, the rate of creep at any time depends on the strength at that time, and the total creep should be calculated by means of an integral, with respect to time, containing a variable term for the stress-strength ratio. Such integration, preferably graphical, is tedious. Moreover, the rate of creep is greatest immediately on application of load, and the strength at that time will therefore be of paramount importance. Some indication of the influence on long term creep of strength at a time subsequent to the application of the load can be obtained from Fig. 4, which shows the relation between creep after 120 days under load and strength of mortar at that time.

Note in this connection that Glanville[23] found that from the age of several months onward the rate of creep is independent of age at loading. This is interpreted to mean that while early creep is largely affected by conditions at that time, once the strength of the specimens becomes approximately constant, their rate of creep is the same.

MAGNITUDE OF CREEP FOR DIFFERENT STRESSES AND FOR DIFFERENT MIX PROPORTIONS

The tests described above were limited to two different stresses. To check the general validity of the suggested creep versus stress-strength ratio relationship, mortars made with two cements—one of high creep, the other of low creep for the same stress—were subjected to a wide range of stresses. This was done both for the dry and the humid storage conditions.

The results of these tests are collected in Table 5, and referring to that table and also to Fig. 2, it can be seen that the same general creep versus stress-strength ratio relationship holds good. The same creep can be expected in mortars made with different cements, and consequently of different strength, if they are loaded to the same proportion of their strength.

So far, the effects of varying the cement and the stress have been considered, but in all these tests the mix proportions were kept constant. It was not

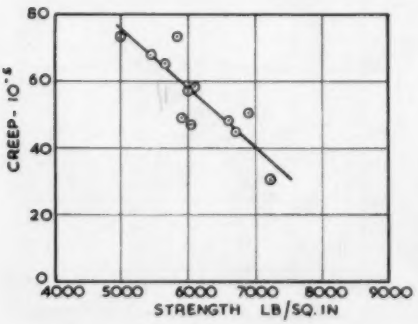

Fig. 4—Relation between creep after 120 days under load and the strength at that time for mortars made with different cements and stored under humid conditions. See Table 3

TABLE 5—CREEP OF MORTARS SUBJECTED TO DIFFERENT STRESSES

Cement No.	Storage condition	Sustained stress, psi	Stress as a percentage of strength	Creep (10^{-6}) after a period under load		
				20 days	60 days	80 days
2	Humid	720	19	18	26	27
		1430	38	30	44	51
		2150	57	50	65	69
		2860	75½	70	87	90
7		830	15½	17	24	26
		1430	27	21	30	32
		1570	29½	26	34	38
		2150	40	32	41	42
		2650	49½	38	51	55
2	Dry	360	13	42	57	58
		710	26½	71	98	100
		1070	40	102	135	139
		1430	53	113	152	162
7		360	16	17	24	24
		720	32	31	41	42
		1070	48	47	60	61
		1430	64	54	76	79

intended in the present investigation to study the effect of richness of mix or of mix proportions in general. Some attempts in that direction have been made in the past, notably by Lorman[24] and by Davis, Davis, and Brown,[25] but considerably more work is required before these factors can be expressed in relation to the strength effects.

Such tests can be done only with graded aggregate since aggregate of the Leighton Buzzard standard sand type permits only a small range of aggregate-cement and water-cement ratios. For this reason, only a few different mixes were used in the present tests, the water-cement ratio varying between 0.40 and 0.52, and the aggregate-cement ratio between 3.0 and 3.7. Table 6 gives a summary of the mix and creep data, and it can be seen from Fig. 2 that within the range of the mixes used, the general creep versus stress-strength ratio relationship holds good.

It is important to note that the regression line in Fig. 2 was calculated for specimens of standard mix proportions and subjected to the same stress only.

TABLE 6—CREEP OF MORTARS OF DIFFERENT MIX PROPORTIONS STORED UNDER HUMID CONDITIONS

Cement No.	28-day cylinder strength, psi	Approximate proportions		Sustained stress, psi	Stress as a percentage of strength	Creep (10^{-6}) after a period under load		
		Water-cement ratio	Aggregate-cement ratio			20 days	60 days	80 days
7	5350	0.40	3.0	1430	27	21	30	32
	5350	0.40	3.0	2150	40	32	41	42
	3610	0.44	3.3	1430	40	38	44	45
	3350	0.47	3.4	2150	64	57	72	76
	2860	0.52	3.7	1430	50	39	50	52
4	5280	0.40	3.0	1430	27	22	30	33
	5280	0.40	3.0	2150	40½	37	47	50
	3930	0.44	3.3	2850	72	65	86	93
	2650	0.52	3.7	720	27	21	28	30

Thus the effects of varying either mix proportions or applied stress were not included in determining the "best" line. And yet, it can be seen that the points obtained in the present series lie very near the original regression line. This indicates that the effect of changing strength by use of a different cement or by varying the water-cement ratio, is the same as far as creep is concerned. Likewise, for a given stress-strength ratio creep is the same regardless of how stress and strength have been varied, provided their ratio is constant.

In view of the above, it seems that strength of the specimen subjected to a sustained load is a primary factor in the magnitude of creep; for constant mix proportions and for a constant stress, creep is inversely proportional to the strength of the cement used. For different stresses the relationship may be expressed in a more general form: *creep is directly proportional to the ratio of applied stress to the strength of the cement mortar, (the stress-strength ratio), regardless of the identity of the cement.* For given mix proportions this seems to be a fundamental property of mortars, and it holds good regardless of the storage conditions, as long as the mortar is free from shrinkage. The fact that the magnitude of creep is an inherent property of all mortars loaded to the same fraction of their ultimate strength, and is independent of the ambient vapor pressure (as long as it is equal to that of the cement gel before loading) is of great importance.

From the above data it appears likely that the relationship between creep and the stress-strength ratio is a fundamental one, although its validity cannot as yet be assumed to extend outside the range of the mixes and the type of aggregate tested.

It seems that strength of mortar or concrete, as measured on a test specimen, predicts one more property of the material—in this case creep—and hence justifies the belief that high strength concrete is, ipso facto, high quality concrete. It must not be forgotten, however, that high creep may often be beneficial.

INFLUENCE OF FINENESS OF CEMENT ON CREEP

Earlier tests on the influence of fineness of cement on creep have been reviewed by the author,[3] and the important, though indirect, effect of gypsum content relative to fineness was noted. A cement with a gypsum content below the optimum would result in an early formation of tricalcium aluminate hydrate and a consequent increased creep.[26]

Provided a cement is properly retarded it would be reasonable to expect fineness to affect creep only in so far as it affects the rate of hardening of the cement paste. An attempt to verify this in the present investigation has not been wholly successful. Two cements were reground to increase their specific surface by some 300 and 800 sq cm per g each, but at 28 days there was no significant difference in strength. The creep data are listed in Table 7, from which it can be seen that the creep is not appreciably affected by cement fineness.

TABLE 7—CREEP OF MORTARS MADE WITH CEMENTS OF DIFFERENT FINENESS

Cement No.	Specific surface, sq cm per g	Storage condition	28-day cylinder strength, psi	Sustained stress, psi	Creep (10^{-6}) after a period under load		
					20 days	60 days	80 days
2	3810	Humid	5350	2150	50	65	69
2A	4090		5400		50	72	77
2B	4670		5120		45	58	61
7	3430		5350		32	41	43
7A	3760		5400		32	39	41
7B	4250		5120		28	37	38
2	3810	Dry	2690	1430	113	152	162
2A	4090		2740		109	146	156
2B	4670		2930		135	179	190
7	3430		2220		54	76	78
7A	3760		2130		57	79	85
7B	4250		2740		42	63	69

One of the cements which had been reground, No. 2, showed what appeared to be abnormally high creep under dry storage conditions. The Cements 2A and 2B (reground) showed the same high creep thus indicating that whatever the cause of the high creep it was not affected by an increase in fineness.

It would be instructive to load similar mortar specimens at an early age, when the influence of the fineness on strength would be more marked. It is believed that the resulting creep would still conform to the general creep versus stress-strength ratio relationship.

As far as the fineness of the original 11 cements is concerned (Table 1) no influence per se is apparent. It might therefore be tentatively concluded that, within the range tested, fineness is significant only in so far as it affects the rate of hardening of the cement paste.

OTHER FACTORS RELATING TO CREEP

Data of Tables 3 and 4 show no correlation between creep and the ratio of the strength of the mortar at the time of the application of the load to the strength at the time of release of load. Thus relative gain of strength is not in itself a factor in creep, although absolute gain in strength during the period when the specimen is under a given stress modifies slightly the creep-time curve. The relation between creep and strength at the time of load release is shown in Fig. 4.

In connection with considerations of the strength ratios it is interesting that for the humid cured mortars made with the English cements, the ratio of strength at 148 days to the 28-day strength lies between 1.25 and 1.44, a comparatively narrow range. Likewise, the ratio of the creep after 120 days under load to the creep after 20 days under load for the same cements and storage conditions is between 1.23 and 1.46. The creep ratio seems to decrease as the strength ratio increases but this relation is only approximate and is not believed to be of help in predicting long term creep from short term

creep data. The small range of values of the creep ratio may, however, be of use in forming an estimate of the magnitude of creep at later ages. The small variation in the rate of creep after 20 days under load is shown by the fact that from that time onward the graphs of creep versus logarithm of time under load plot as straight lines of nearly the same slope for mortars made with different cements.

Compared with humid storage conditions, the increase in the strength of dry specimens between 28 and 108 days is much lower—between 1.03 and 1.15 expressed as a ratio, and the ratio of the creep after 80 days under load to the creep after 20 days is higher—between 1.42 and 1.68. The American cements, on the other hand, show a considerably higher gain in strength. This is of interest since their C_2S content varies between 10 and 52 percent; little gain in strength would be expected in the cement with the lowest C_2S content. Thus the C_2S content does not seem to be the paramount factor in the gain of strength at later ages, at any rate for dry storage conditions.

The range of the C_2S contents of the English cements is comparatively smaller—between 25.3 and 40.0 percent—but even so the variation in relative gain in strength after 28 days seems very small. It would thus appear that while the chemical composition of cement largely determines the strength of mortar of given mix proportions, the gain in strength at later ages may not be a function of the C_2S and C_3S contents. These data would indirectly support Powers'[18] observation that all compounds in cement hydrate at the same fractional rate, in contradiction to the classical theories.

Returning to the creep versus stress-strength data given earlier, it may be observed that for the dry condition, the postulated relationship between creep and the stress-strength ratio is not followed as closely as for the humid condition. This may be due to the influence of some other, as yet undetected, factor. In particular, Cements 9, 10, and 11 exhibit lower creep than would be expected; these cements are all of U. S. origin, and possibly there is some common factor in their history which is responsible for this apparent lowering of creep in the incompletely hydrated state. There may be a connection between these creep characteristics and the fact that the same cements show considerably higher gain in strength after 28 days than do any of the English cements. However, in the present state of our knowledge, it is not possible to identify the cause of either the low creep or high strength gain of these mortars when stored dry.

Behavior of Cement 2

A far greater anomaly is represented by the creep of Cement 2 under similar conditions. Data have been carefully checked, and there is no doubt that mortar made with Cement No. 2 and stored dry exhibits exceptionally high creep. Mortars made with Cements 2A and 2B, obtained by regrinding of Cement 2, show similarly high creep. Likewise, creep values for this cement

subjected to various stresses all conform to the proportionality of creep to applied stress for constant strength, but are all far in excess of the values for all other cements tested.

It is significant that Cement 2 has also exhibited a high shrinkage. While all other mortars under dry conditions shrank less than 8×10^{-5}, the shrinkage of No. 2 (as well as 2A and 2B) was 13×10^{-5} at 58 days and 23×10^{-5} at 88 days; these ages correspond to 30 and 60 days under load, respectively. Powers,[36] referring to work by Verbeck, points out that shrinkage caused by carbonation, as well as by drying, increases with the alkali content of cement, and it is possible that creep is similarly affected.

It must be repeated that the present investigation was not concerned with all the factors which may affect creep, but solely with the role of the cement. It is possible that some other factors influence creep in so far as they affect the strength of the mortar or concrete in a manner similar to the influence of the quality of cement. Creep and recovery tests are, par excellence, time tests, and consequently a long time is required both to collect and to analyze the data. It is hoped that with the aid of the apparatus which has been developed further data on the influence of various factors in creep will be obtained.

CREEP VERSUS STRESS-STRENGTH RATIO RELATIONSHIP AND THE RESULTS OF OTHER INVESTIGATORS

It should be stressed again that the suggested relationship between creep and the stress-strength ratio is believed to hold good for a small range of mix proportions only and—since no experiments indicating otherwise have been made—for specimens of similar shape and size. Nevertheless, the relationship, if true, should hold good for suitable groups of test results obtained by other investigators.

The suggested proportionality of creep to the stress-strength ratio implies, ipso facto, that for a given mix the creep is proportional to the stress applied. In the past this has been believed to be true only for stresses below about 30 percent of the ultimate, i.e., for the usual range of working stresses.

Data obtained in the present investigation suggest that the proportionality holds good up to 75 or even 90 percent of the ultimate. It seems, however, that the creep-stress function does not necessarily vanish at the origin, and, therefore, the specific creep (per unit stress) may not be constant. This threshold value may possibly be purely a matter of the least measurable strain.

Many data obtained in the numerous creep investigations do not include information on the strength of the specimens but only a description of the "1:2:4 mix" type; in those cases no verification of the suggested relationship can be obtained. On the other hand, Chatterjee[29] recently measured the creep of similar concrete specimens subjected to a sustained stress of between 55 and 83 percent of the ultimate compressive strength. He calculated specific creep at various stresses, found it to increase with stress, and con-

cluded that creep was proportional to some higher power of stress. This author has replotted Chatterjee's data (Fig. 5), and found that for ages greater than 3 days creep is directly proportional to stress in the range of 55 to 83 percent of the ultimate, but that an extrapolation of the straight line would indicate no creep at very low stresses. This may be caused by the fact that creep during loading (a period of a few minutes) is not measured: such creep would represent the major part of the total creep at low stresses. That some creep takes place even at low stresses is shown by the fact that the stress below which the extrapolated curves of Chatterjee's show no creep, is smaller the greater the time under load. Similar, although very small, change is exhibited by the curves of the present investigation.

The data of Davis, Davis, and Hamilton[30] summarized in Table 1 of their paper show, likewise, that creep is proportional to the stress applied, and the lines of creep versus stress plotted from their data indicate no creep for a stress lower than about 120 psi. The stress-strength ratio in their tests probably does not exceed 50 percent.

Shank[31] measured short-time creep of specimens subjected to stresses between 85.5 and 90.5 percent of the ultimate. He found the specific creep to increase with stress, but a replot of his data on the basis of total creep versus stress-strength ratio shows a linear relationship (Fig. 6). This line extended would indicate no creep at a stress as high as 80 percent of the ultimate; it is

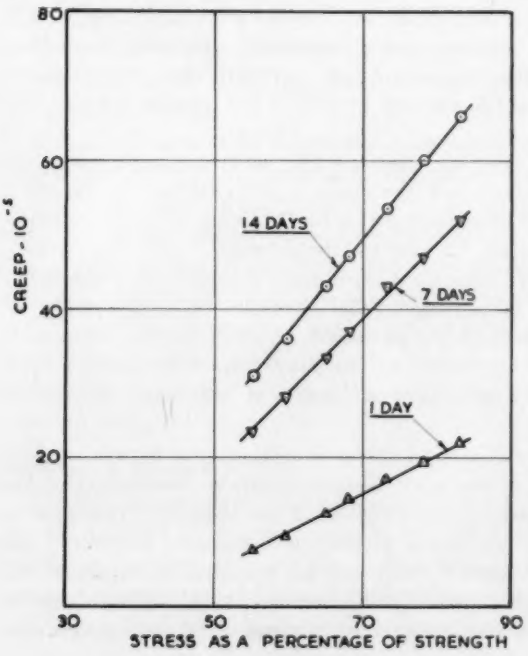

Fig. 5—Chatterjee's data[29] for creep after different periods under load recalculated to illustrate the creep versus stress-strength ratio relationship

probable, and indeed, indicated by some of Shank's data that another straight line relationship for creep versus stress holds good for stresses below about 85 percent of the ultimate. The existence of two distinct creep-stress relationships is confirmed by some of Evans'[32] data. Thus, it seems, a simple relationship between creep and stress holds good up to a high value of the stress-strength ratio, and the relation is more accurate for long term creep, which is of more practical interest. Stresses above some 85 or 90 percent of the ultimate are rarely met with, and generally lead to failure after a time under load.

For short periods under load—of the order of minutes—creep seems to be proportional to a higher power of stress. This is indicated by some of Chatterjee's results,[29] and more definitely by those of Glanville and Thomas,[33] and of Evans.[32].

Washa and Fluck[34] measured creep of concretes made with three different cements: ordinary portland, rapid hardening portland, and portland pozzolan. They observed that concrete made with the last-named cement exhibited creep for the longest time and still showed a considerable increase in creep between 2 and 10½ years. For the portland cements without pozzolan there was little increase in creep after 2 years under load. From a re-examination of their data it can be seen that the portland pozzolan cement concrete which showed a large increase in creep showed no increase in strength between the time of application of the sustained load and 10½ years, while strength of the portland cement concretes increased considerably. This is of interest in view of the suggestion made earlier in this paper that creep is a function of the strength at all times under load. It may also be recalled that Glanville and Thomas[33] observed a constant rate of creep for long periods under load in the case of aluminous cement concrete; the strength of concrete made with that type of cement is known to increase little with time.

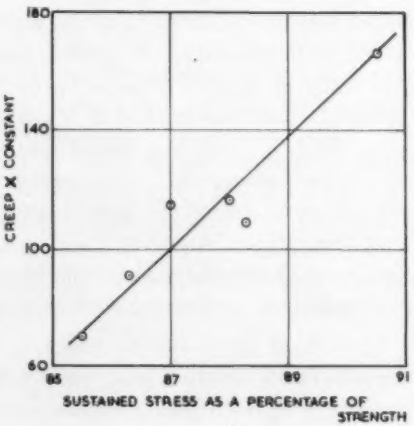

Fig. 6—Shank's data[31] for creep after 3 hr under load recalculated to illustrate the creep versus stress-strength ratio relationship

Returning to Washa and Fluck's[34] tests on portland cement concretes, creep after 2 years under load can be seen to be sensibly constant for a constant stress-strength ratio (based on the strength at the time of application of the load) for any one mix. It seems, however, that there is some inconsistency in the 28-day strength data for one of the water-cement ratios used (0.66), and for that reason the relation between creep after 10½ years under load and the strength of mechanically compacted actual creep specimens, as determined at 11 years, was investigated and showed good agreement with the postulated creep versus stress-strength ratio relationship. It may be recalled that such a relationship, based on strength at the end of the period of creep measurement, is only approximate.

During the writing of this paper a translation of a recent Russian paper on creep was received[35] dealing with creep of concretes made with different cements and subjected to stresses representing the same proportion of the ultimate strength. As only three cements were investigated the general conclusions regarding the influence of the individual compounds seem rather bold, but it is interesting to note that while the specific creeps of the three cements varied widely (2.3, 5.2 and 3.3×10^{-6} per kg per sq cm at 20 days) the total creeps for the same stress-strength ratio were of the same order for the three cements respectively: 30, 33, and 30×10^{-5} after 20 days, and 124, 153, and 122×10^{-5} after 660 days under load. Their strengths at 28 days were 257, 124, and 180 kg per sq cm, respectively. The divergence at later ages may well be due to the different rates of gain of strength with time for the different cements, thus altering the effective stress-strength ratio. Individual points in the creep-time graphs show considerable scatter, and it is therefore probable that the experimental error would account for some of the variation in creep between concretes made with different cements. Allowing for this, creep seems to be reasonably constant for a constant stress-strength ratio regardless of the inherent strength of concrete.

An indirect confirmation of the influence of the stress-strength ratio on creep may be obtained from the test data of Davis, Davis, and Hamilton.[30] They found that creep of concrete loaded at 28 days and stored dry was twice the creep of similar concrete subjected to the same stress but stored wet. When the load was applied at 3 months the corresponding ratio of creeps was equal to 3. It was also found that for dry storage creep was little affected by the age at loading. These phenomena may be explained by the fact that the concrete stored wet gained strength faster than the dry concrete, and that the ratio of strength of wet concrete to that of dry concrete would be greater at 3 months than at 28 days. For that reason the inverse ratio of corresponding creeps would be greater, too.

In view of the above discussion it seems probable that for periods exceeding several days under load creep is proportional to the stress-strength ratio regardless of the identity of the cement, at any rate for mortars and some

concretes. At high stresses—of the order of 85 percent of the ultimate—creep may increase at a greater rate, and indeed, may lead to failure after a certain time under load.

No data, other than those presented in this paper, are available to check the general relationship between creep and the stress-strength ratio for *any* portland cement. Indeed, a comparison of normal portland cement concrete with portland blast furnace cement concrete made by Ross[4] shows that the latter exhibits an inherently higher creep. The same applies to portland pozzolan cement concrete tested by Washa and Fluck.[34] It must be remembered, however, that the relationship postulated in this paper applies only to the range of cements tested, i.e., to cements made from portland cement clinker without further addition of material other than gypsum.

CONCLUSIONS

Tests described in this paper cover only a narrow range of mortar mixes, and so conclusions drawn from them cannot, without further experimental evidence, be assumed to apply to all mortars. Nonetheless, it is possible to state that the strength of mortar as a function of the identity of cement is a primary factor in creep. For a given applied stress, creep is approximately inversely proportional to the strength of mortar at the time of application of load.

For different applied stresses, in mortars of the same strength, creep is proportional to the stress applied. This proportionality seems to hold good over a range of stresses between 20 and 80 percent of the ultimate strength, i.e., up to a value considerably higher than hitherto believed. It appears thus that for mortars of fixed mix proportions but made with different cements and subjected to different stresses the magnitude of creep can be expressed as a linear function of the ratio of applied stress to strength of mortar. This stress-strength ratio is a basic factor in creep.

These conclusions have been drawn from tests on mortars cured and stored at 95 and 32 percent relative humidity, and are corroborated by the author's[20] earlier tests, in which the relative humidity was 75 percent. It seems, therefore, that the humidity of the surrounding medium does not affect creep, provided mortar has reached hygral equilibrium with this medium before the application of the load, so that the effects of shrinkage are not superimposed on creep. This fact is of interest since in the past creep has often been regarded as a function of the ambient humidity. The effects of humidity should not be neglected but its influence on creep seems to be only indirect, mainly in so far as curing conditions affect the strength of mortar. Humidity would also be of consequence in the possible interaction between shrinkage and creep.

Pilot tests on mortars of varying mix proportions indicate that the same creep versus stress-strength ratio relationship holds good. It is not possible,

however, to state whether richness of the mix is also a factor in creep. This can be established only from tests with a graded aggregate on a wider range of mix proportions.

Tests on some of the cements reground to a higher fineness indicate that fineness is a factor in creep only in so far as it affects mortar strength. This conclusion requires confirmation over a wider range of finenesses, particularly for mortars loaded at an early age, when the effect of fineness on strength is greater.

The relation between long term creep and creep after 20 days under load appears to vary comparatively little, and there are indications that this variation in creep and the increase in strength with age are related. This would be in agreement with the suggestion that creep is affected by the strength of the specimen throughout the period during which it is subjected to a sustained load.

A re-examination of test results of many other investigators shows a general agreement with the proposed creep versus stress-strength ratio relationship. It is possible, however, that in short-time tests the coefficient of proportionality between creep and the stress-strength ratio is higher in the upper range of values of this ratio. Further tests are needed to clarify this point.

The significance of the proportionality between creep and the stress-strength ratio lies in the fact that, since the instantaneous elastic strain is also approximately proportional to the same ratio, the total strain seems to be greater the nearer the applied load is to the ultimate strength of the mortar. As a corollary, it would seem that the same strain corresponds to the failing stress in all mortars. This is the basis of the ultimate strain theory—a conception of failure increasingly prominent in design work of recent years. Data obtained in the present investigation indicate that the ultimate compressive strain of mortars of the type used is of the order of 0.002. It is realized that this figure is obtained by extrapolation, which, particularly in the region near the ultimate, is a delicate and somewhat speculative operation. It may be interesting to remember that specimens subjected to a sustained stress a little below their strength are known to fail after a time under load. This failure is probably caused by the increase in strain due to creep, the strain in the specimen finally reaching the ultimate strain value.

As far as other practical applications are concerned, the knowledge that a lower stress factor leads to lower creep is of vital interest in prestressed concrete work, as well as in many other types of structures in which the effects of creep are harmful. It is realized that the work described deals primarily with mortar but the results obtained can be logically expected to indicate the trends and the relative significance of the diverse factors in the creep of concrete.

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Title No. 55-63

design and construction of a Modern Parking Garage

By WALTER E. RILEY

HAUNCHED FLAT SLAB construction with round columns was used for a six-level parking garage built in Phoenix, Ariz. Approximately 137 x 200 ft, garage has ramps at a 12 percent grade. Traffic design, structural features, construction practices, and shrinkage cracking are discussed. Camber was introduced into slabs by jacking the forms while they held the finished but unhardened concrete.

■ Owners of valuable property in downtown Phoenix, Ariz., faced with the ever increasing problem of parking for tenants of their three office buildings, decided that a parking structure should be built. A survey was made, and it was decided that 375 stalls were required for the tenants and 60 short time parking stalls for visitors. Since the land available for the garage was approximately 137 x 201 ft a six-level structure was required.

To give the tenants complete freedom of movement it was decided to build a self-parking type of structure. The first floor (Fig. 1) was laid out so that one entrance would control the inflow and outflow of all cars to the basement and the five floors. Since the upper floors are for tenant parking only, with no tickets involved, no appreciable reservoir space was required at the ramp

entrance. The areas immediately to the right and left of the ramp provide sufficient reservoir space for the short time parkers who do use tickets.

For a typical upper floor, stall widths average 8 ft 6 in. and lengths, 19 ft 4 in. Access aisles are 20 ft 4 in. and angle of parking is 60 deg. Unit parking depth is 59 ft. Since the property was only 137 ft 5 in. wide, special permission had to be secured from the city council to extend the floors 3 ft over the property line and over the alley. On the first floor this was not possible, so all stalls are 1 ft 6 in. shorter than on the upper floors. A study of the turning radius of various automobiles indicated a 26-ft design radius would be adequate. Actual operation of the facility has proved this to be true.

Ramps

Ramps are one way, parallel straight type. They incorporate the adjacent parking principle wherein part of ramp travel is on the access aisle. Grade is 12 percent. A typical cross section is shown in Fig. 2. Each end of every ramp is blended into the floor by using a 6 percent grade (Fig. 3).

Interconnecting features

A footbridge crosses the west end of the alley at the third floor and connects with the second floor of the 14-story Luhrs Tower. At the northeast corner an S-shape bridge spans the alley to the second floor of the 11-story Luhrs Building. This bridge spans over 25 ft and its ends are offset later-

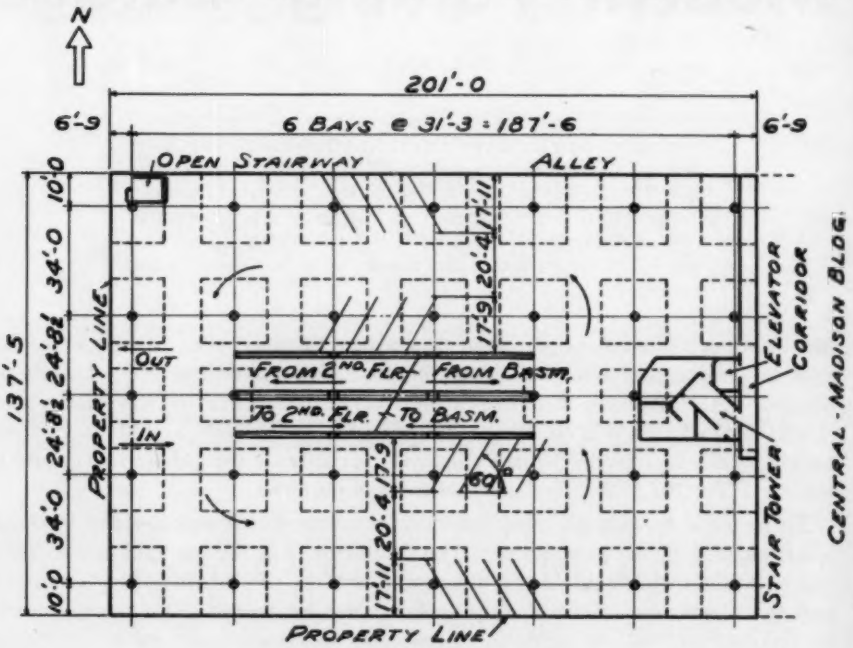


Fig. 1—First floor plan

WALTER E. RILEY, consulting engineer, Phoenix, Ariz., holds the distinction of being a licensed civil, structural, and mechanical engineer. He writes authoritatively on the subject of flat slab construction having successfully employed this method many times. Mr. Riley has written a number of technical articles and, having conducted considerable research in thin shells, holds a number of patents on this method of construction.

ally by practically the same amount, thus presenting some rather unusual problems of torsion. The elevator and stair tower serves both the garage and the adjacent Central Madison Building, with doors opening into the 6 ft 9 in. wide strip used as a corridor for the Central Madison Building. As shown in Fig. 4, the second and third floors of the garage were off-set vertically at the east end so as to be flush with the floor and roof slabs of the adjacent building.

STRUCTURAL DESIGN

The structure is haunched flat slab construction with round columns. Rather unusual story heights (Fig. 5) make feasible the use of the structure as a future office building since the ramps are structurally independent of the floors and may be cut out thus giving an interior lightwell. As shown in Fig. 1, typical floor bays are 31 ft 3 in. by 34 ft, which as far as is known appear to be the largest spans ever used for this type of construction. The north and south floor edges cantilever 13 ft beyond the columns.

For the floors, 3750 psi minimum concrete was used. Live load was 80 psf placed to give maximum negative and positive moments. The design was by elastic analysis in accordance with the ACI Building Code (ACI 318-56). Moment distribution was used with stiffness factors and carry-over factors for the two way variable moment of inertia floor slabs determined by the column analogy method.

In an effort to reduce plastic flow of the concrete in the underside of the slab at the columns and the resulting slab deflection, positive steel in the

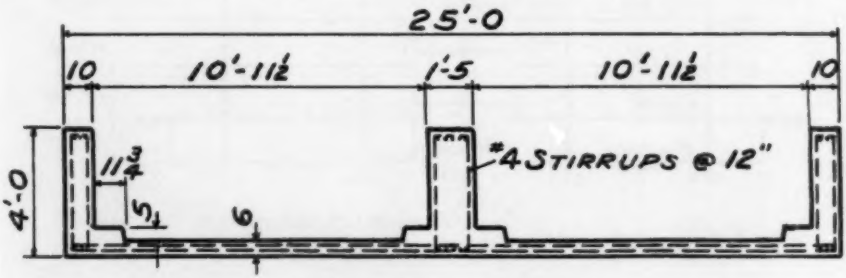


Fig. 2—Typical ramp section

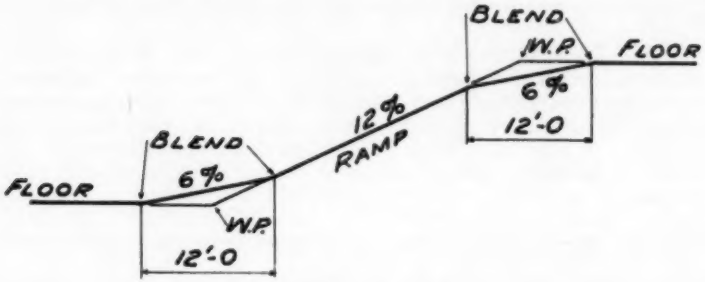


Fig. 3—Ramp blending

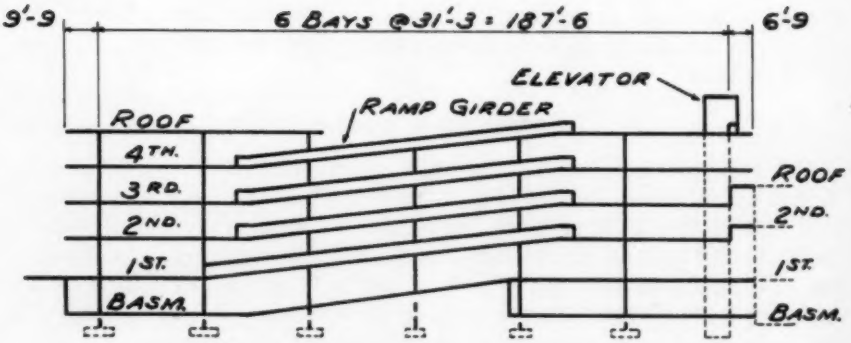


Fig. 4—Longitudinal section

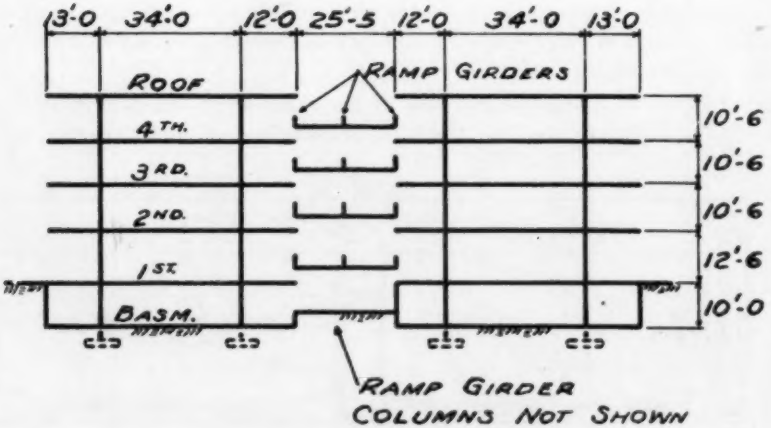
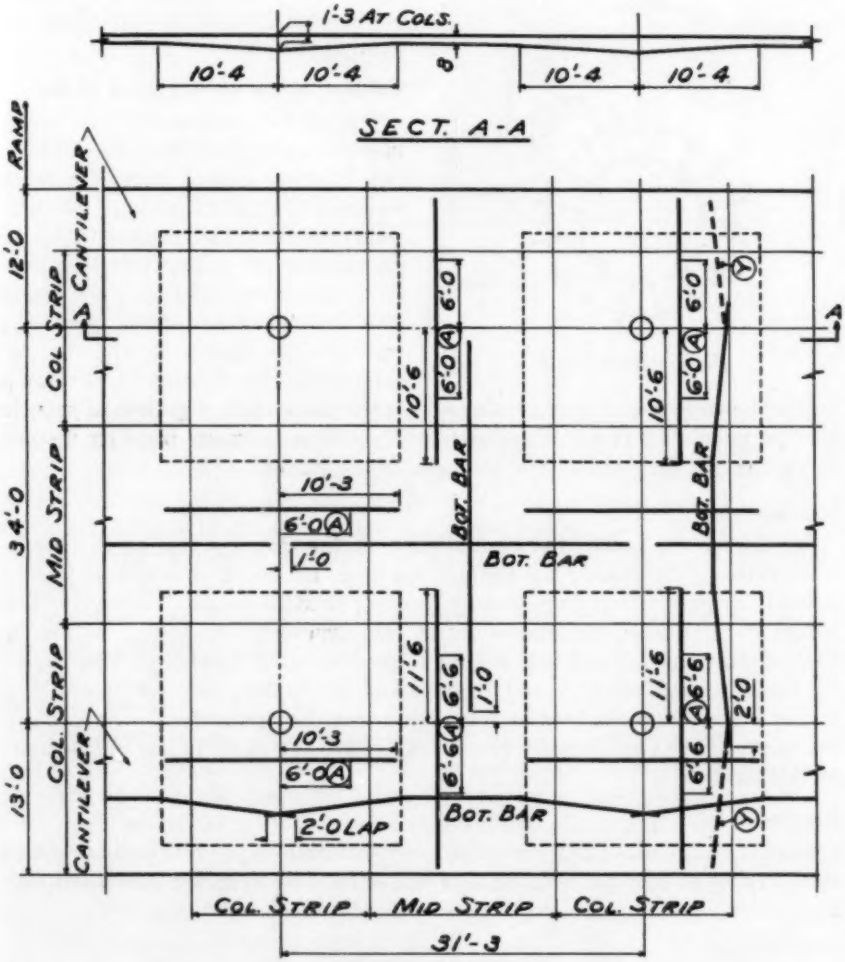


Fig. 5—Transverse section

column strips was extended past the column and lapped as shown in Fig. 6. At the cantilevers, every third bottom bar was extended to the end of cantilever to reduce plastic flow of the underside of the cantilever with its resulting deflection. In the column strips, two-thirds of the top steel was concentrated in the middle half of strip to provide for the peak stresses occurring at columns.



(A) ALT. TOP BARS STOP HERE.
(Y) EVERY 3RD BOT. BAR EXTENDS TO END OF CANTILEVER

Fig. 6—Typical floor steel

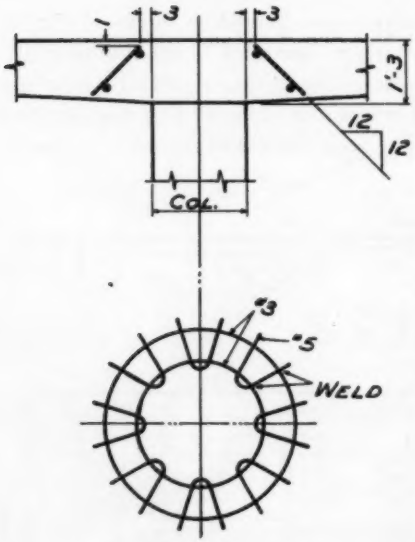


Fig. 7—Shear head

Columns were made of 5000-psi concrete, and vary from 20 to 30 in. in diameter. Shear heads were used for all columns 26 in. or less in diameter as shown in Fig. 7.

Footings are spread type, designed for a maximum soil pressure of 10,000 psf on the coarse sand and gravel bearing stratum.

Column below central ramp girder

One of the unusual features is the first floor column at the bottom of the central ramp girder. Since the ramp girder is only 17 in. wide it was necessary to limit the column to 17 in. diameter so as not to interfere with ramp clearances. To carry the heavy load from the floors above, steel tube was used as shown in Fig. 8. Incidentally, the stirrups in the ramp girder shown in Fig. 2 were continuous for the full length of girders to provide for the possibility of severe axial tensile shrinkage stresses building up and impairing the diagonal tensile strength of the girder.

Stair opening

An interesting problem presented itself at the stair opening in the northwest corner. Ordinarily an opening as large as this and common to two column strips is completely framed to carry load to columns; however, this would have spoiled the clean lines of the structure. As shown in Fig. 9, part of the stair load and that portion of floor north of the column was carried by the floor cantilever *A* and then picked up by the floor cantilever *B*, a rather heavy concentration of reinforcing steel being involved. Apparently the approach was satisfactory as no cracks have shown up in any of the floors at this location.

Drainage

Roof drainage was taken care of by a conventional type of concrete drainage slab sloping to four drains; the slab was separated from the structural slab by a three-ply waterproof membrane using slip sheet technique.

CONSTRUCTION

No unusual features were encountered in the basement. There is no water table problem in this location so the excavation was made easily. As mentioned previously, all footings rest on a coarse sand and gravel stratum which extends for a depth of over 100 ft.

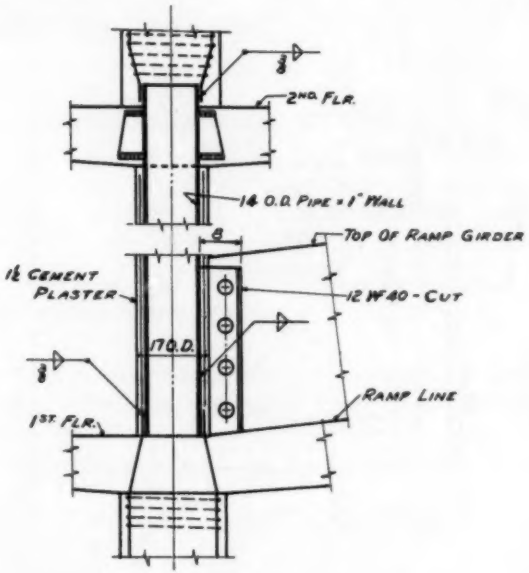


Fig. 8—Special column at ramp entrance

Floors were not troublesome. Haunches were kept constant size for all five floors so that forms were reused with the only modification being a collar piece at the column to accommodate the reduction in column size in upper stories. Forms for columns were heavy paper tubes which were left on for several weeks and affected an excellent cure for the concrete. The floors were double shored to support the weight of the wet concrete and the construction equipment.

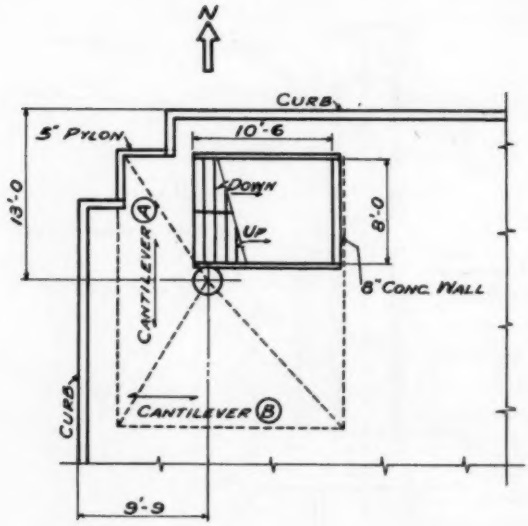
Total floor area of approximately 175,000 sq ft was built at a cost of $3.28 per sq ft, or $1320 per stall.

Floor camber

Ordinarily, flat slab floor forms are built to a cambered outline as shown in Fig. 10 to provide for dead load deflection of slab. The finishing screeds are then set from the cambered form, thus giving a convex concrete surface. This is no particular problem with the usual flat plate or a slab with drop panels since it is easy to warp the form surface. With tapered haunches it becomes much more complicated as the haunch does not lend itself to warping.

Using shores of the adjustable type as shown in Fig. 11, it was specified that the floor forms would be built with no camber and after finishing, the central portion of each bay would be raised approximately 1½ in. with the amount of rise decreasing as the haunches were approached. Theoretically, 2 in. should have been used to allow for 1 in. sag of the supporting floor below and 1 in. sag of bay when shores were removed. However, it appeared

Fig. 9—Stair opening



that 1½ in. was the limit, as the forms would be subjected to too much racking. After placing and finishing, the floor was then raised by means of a special jack attached successively to each shore thus lengthening the shores and raising previously finished floor. This method, utilizing the period between initial set and final set of concrete, gave a satisfactory floor with relatively small deflections, although not quite as uniform as a formed camber.

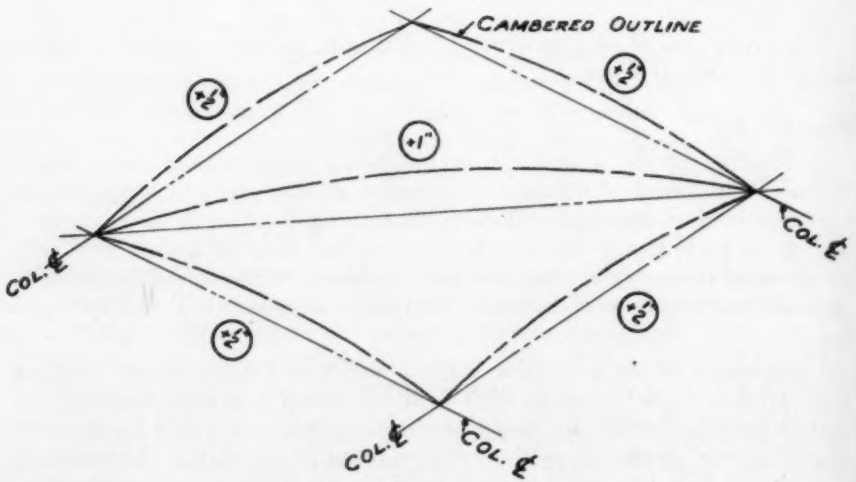


Fig. 10—Camber usually built into forms

ARCHITECTURAL TREATMENT

Exterior edges of the slabs and the corner pylons were wet rubbed with a carborundum stone to give a smooth texture and uniform color. No ties were permitted to extend through any members specified as architectural concrete; thus the corner pylon forms were of special design. In one instance, the form bulged out about 1½ in. which necessitated chipping and grinding. Vinyl was added to the rubbing grout which also had a percentage of white cement in addition to the portland cement so as to give the same color as the rest of the concrete.

CRACKING PROBLEMS

Plastic shrinkage cracking

Phoenix is in a desert area of the country and is subjected to extremely high temperatures and low humidity. The third and fourth floors of the garage were placed during midsummer when the temperature was up to 112 F and humidity was down to 7 percent. This of course resulted in some plastic shrinkage cracking. Rapid drying of the surface is usually thought to cause these cracks; however, these surfaces were actually quite moist and protected by a heat reflecting curing compound applied shortly after finishing.

The hairline cracks appeared mainly around the columns as shown in Fig. 12 where the steel is concentrated near the top surface of slab. They showed up a few hours after finishing even in areas where the bleeding was copious; so it would appear that there is more to plastic shrinkage than rapid evaporation of water from the concrete surface.

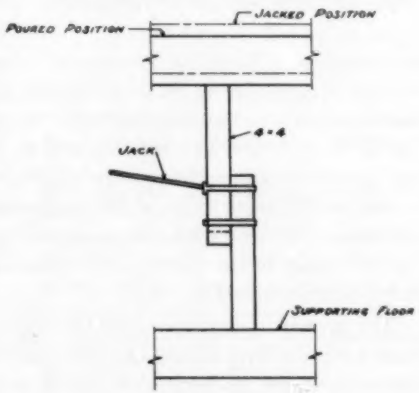


Fig. 11—Shore

Drying shrinkage

Cracking due to volumetric shrinkage—drying out of the concrete—did not appear for several weeks. A typical pattern for the first floor is shown in Fig. 13. The pattern shown is quite reasonable. Along the previously placed basement wall, the slab is under almost total restraint to shrinkage; therefore the slab cracks sufficiently to equal the shrinkage

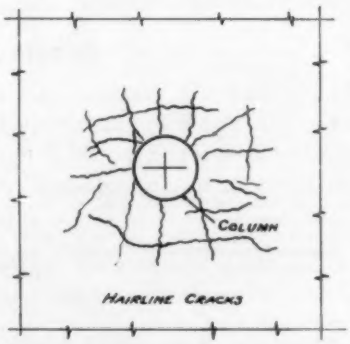


Fig. 12—Plastic shrinkage cracks

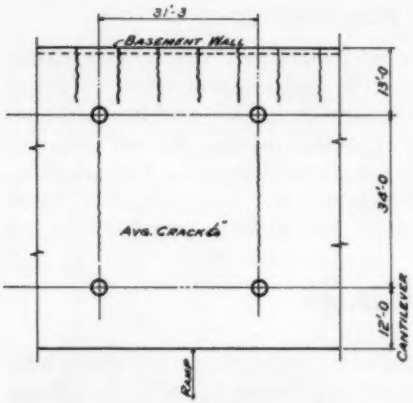


Fig. 13—First floor shrinkage cracks

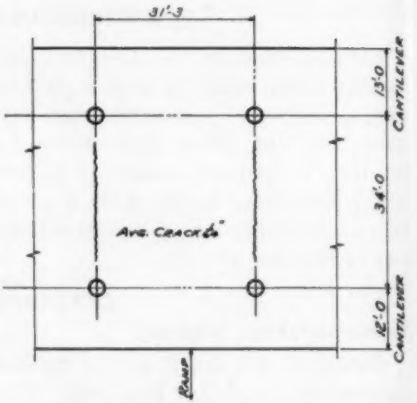


Fig. 14—Typical floor shrinkage cracks

contraction. Even at transverse construction joints the pattern did not change much. Based on four 1/64-in. cracks in a distance of 31 ft 3 in., the net coefficient of shrinkage is 0.00016. On the column line in the central portion of the 34 ft span, the slab reduces to 8 in. thick; it is natural to expect the shrinkage cracks to concentrate here since the negative bending stresses of the slab action are additive to the shrinkage stresses. Relative rigidity of the supporting columns compared with the basement wall is indicated by reduction of the number of cracks in the 34-ft span. The basic pattern of cracks for the remaining floors is shown in Fig. 14.

The writer is confident that the day will come, perhaps in the not too distant future, when we will not have the present problem of shrinkage cracking. If the cement technologists could develop an economical cement additive which will swell as the mixing water is reduced by normal evaporation and hydration, our troubles would be over. The Russians apparently have been doing some work on this problem* but I hope we do not have to wait for them to find the complete answer.

CONSTRUCTION CREDITS

The author was architect and engineer for the parking structure. Contractor was T.G.K. Construction Co. of Phoenix; Ken Hill was the general superintendent. The garage is owned by Luhrs Properties, Inc.

*Mikhailov, V. V., "New Developments in Self Stressed Concrete," *Proceedings*, World Conference on Prestressed Concrete, San Francisco, 1957, pp. 25-1–25-12.

Received by the Institute Oct. 2, 1958. Title No. 55-63 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 9, Mar. 1959 (Proceedings V. 55). Separate prints are available at 50 cents.

American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by June 1, 1959, for publication in the Part 2, September 1959 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Competition for the design of the Weinland Bridge, Switzerland

P. W. Abeles, *Civil Engineering and Public Works Review* (London), V. 53: Part 1, No. 621, Mar. 1958, pp. 293-296; Part 2, No. 622, Apr. 1958, Part III, No. 623, May 1958, pp. 554-556

Reviewed by Gunhard-Aestius Oravas

An excellent survey of all designs submitted in the competition. Individual types of bridges are critically reviewed and construction of the winning design described.

Continuous composite girder (in Japanese)

Yoshio Tachibana and Kazuo Kondo, *Journal*, Japan Society of Civil Engineers (Tokyo), V. 43, No. 12, Dec. 1958, pp. 37-42

Reviewed by Kiyoshi Okada

Reports briefly two of the continuous composite girder bridges recently built in Japan: one is a two-span bridge with span length of 20.8 m and the other a three-span bridge with span lengths of 12.85 m, 15.00 m, and 12.85 m. Presents some results of tests of model girders.

Design

Reinforced concrete design (Bemessung im Stahlbetonbau)

Supplement to the 6th Edition of *Specifications of the German Committee for Reinforced Concrete* (*Bestimmungen des Deutschen Ausschusses für Stahlbeton*), Wilhelm Ernst and Son, Berlin, 1958, 57 pp., 6 DM

Reviewed by J. F. Leppmann

Column and beam design formulas and procedures, including tables and graphs, which were selected some time ago from the works of several authors (Löser, Morsch, Pucher, etc.) for the annually published manual, *Betonkalender*, are presented as German Industry Standard DIN 4224. Their exclusive use in connection with Standard Specification DIN 1045 is recommended by the Committee because of their simplicity and clear arrangement and because they require no high degree of arithmetical precision and are easy to check.

Influence of column hoops on compression members (in Spanish)

Minister of Public Works, *Revista de Colegio de Ingenieros de Venezuela* (Caracas), No. 269, Aug. 1958, pp. 8-14

Reviewed by Joseph J. Waddell

According to the author, column hoops perform four principal functions:

1. Absorb the shearing stresses produced by the thrust in the panel points of the structure.

2. To prevent the longitudinal reinforcement from moving.

3. Prevent buckling of the longitudinal bars.

4. Increase the strength of the column in combination with the longitudinal reinforcement, provided they are within certain limits of spacing and definite quantities.

An analysis is made of the limits of spacing to avoid buckling (Item 3) and of the action in increasing the strength of the member (Item 4).

Shanley effect, The (Der Shanley-Effekt)

Ferdinand Schleicher, *Der Bauingenieur* (Berlin), V. 32, No. 12, Dec. 1957, pp. 449-458

Reviewed by Aron L. Mirsky

This paper, on inelastic action of *metallic* columns, is noted here for information purposes, pertaining as it does to a phase of structural engineering which is still in the

A part of copyrighted Journal of the American Concrete Institute, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55). Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

formative stage. It is based on a lecture by the late author in connection with a refresher course in reinforced concrete construction at the Milan Polytechnic, May 29, 1957.

Domes in unreinforced masonry (Les coupoles en maçonnerie non armée)

P. Bouteloup, *Annales des Ponts et Chaussées* (Paris), V. 128, No. 4, July-Aug. 1958, pp. 429-503

Reviewed by Aron L. Mirsky

In the first part of this work, stability of large unreinforced masonry domes is investigated. Certain types found suitable for architectural use are more fully investigated in second part. Advisability of using hoops in certain cases is brought out. Paper is well illustrated, particularly in second part, with photographs, drawings, and analyses of majestic domes on some world famous structures (which, incidentally, shows that the ancients knew a thing or two).

Materials

Note on water penetration inside a particle of clinker (in French)

A. R. Steinherz, *Revue des Matériaux de Construction et de Travaux Publics* (Paris), No. 500, Feb. 1958, pp. 48-51

Reviewed by Phillip L. Melville

A statistical study of depth of water penetration during hydration and resulting heat of hydration, including 11 references.

ASTM standards in building codes

American Society for Testing Materials, Philadelphia, 1958, 1060 pp., $8

This volume, containing over 200 standards in their latest form, is a new edition of the compilation of ASTM standards found in building codes. It includes 18 standards not previously included in the original 1955 edition or its 1956 supplement.

Ordinarily these standards are scattered among the several volumes of the ASTM Book of Standards; therefore, in combining all such reference standards under one cover, this compilation should prove particularly handy.

Topics covered include admixtures, aggregates, asbestos-cement products, brick, building constructions, cement, and clay tile. Also included in the compilation are: concrete; concrete masonry units; copper and copper base alloys; fire tests; flash tests; gypsum; iron; lead and lead alloys; lime masonry mortar; natural building stone; and pipe and drain tile (nonmetallic). Plastics; refractories; soils; steel; testing methods; thermal insulating materials; wood and wood preservatives are among those topics covered.

Hydraulic value of slag from the behavior of the blast furnace (in French)

J. Roquejoffre, *Revue des Matériaux de Construction et de Travaux Publics* (Paris), No. 512, May 1958, pp. 119-135

Reviewed by Phillip L. Melville

Twenty-one samples of slag from a given ore-producing grade of pig iron were ground. Activity of the slag was found related to fineness. A second series of tests was made on 52 samples from one furnace, ground to the same fineness. It was found that the temperature of topping out of the slag determines initial reactivity and that topping out temperature, density and the CaO/SiO_2 ratio are closely related. Chemical criteria were found reliable in evaluating hydraulic properties.

Precast Concrete

Utilization of large precast panels in the construction of residential buildings (in Russian)

G. F. Kuznetsov, *Beton i Zhelezobeton* (Moscow), July 1958, pp. 249-258

Reviewed by D. Watstein

The author states that the method of construction which uses large precast panels as framing members is now entering a new phase of development in the USSR. This method is becoming a means of large scale residential construction, as well as in construction of school, hospital, and other buildings.

At present three new fabrication methods being developed in industrial enterprises are: (1) a process involving vibration and rolling with the forms moving as an endless belt-conveyor; (2) a process in which the individual forms attached to a conveyor pass under a rolling machine; and (3) a process in which a movable rolling machine passes over a stationary form. Thin shell precast constructions are being produced by vibratory

and rolling process in which the thickness of the shells ranges from 0.6 to 0.8 in.

The author stresses the importance of using locally produced lightweight aggregates to combine the economy of lightweight and simplicity of casting by means of solid slab panels.

Cost of reinforced concrete per square foot of living area in panel framed building is significantly reduced compared with conventional reinforced concrete. Author estimates that the cost of all reinforced concrete elements in a precast panel building ranges from 15 to 20 percent of the total cost per square foot of living area.

Concrete pipe handbook

HOWARD F. PECKWORTH, Ameridan Concrete Pipe Association, Chicago, 2nd Edition, 1958, 499 pp.

This second edition brings up to date a wealth of engineering data and technical information pertaining to concrete pipe. Chapters are included on manufacturing, specifications and tests, bedding and backfilling, loads on underground conduits, hydraulic design of culverts, run-off, jacking of pipe lines, etc. Numerous data sheets, tables, and example problems are included as in the first edition.

A new chapter on autogenous healing of concrete is included as well as an additional appendix section on design and installation criteria for reinforced concrete pipe culverts. Pertinent ASTM standards are also reprinted in the appendix.

Fundamentals of concrete crossties (Grundsätzliches zur Betonschwelle)

HERMANN MEIER, *Beton- und Stahlbetonbau* (Berlin), V. 52, No. 6, June 1957, pp. 129-139
Reviewed by VALDIS LAPSINS

Extensive experiments conducted at the Technical College in Munich to study the performance of concrete in use as railroad crosstie material. Main advantages of concrete are reported (1) resistance to weathering, (2) high compressive strength, and (3) variety of shapes. Disadvantages, such as weight, tensile strength, and form adjustment can be fully or partially overcome by means of design and construction. Paper also fully describes theoretical as well as practical considerations in tie loading and design. A word about the concrete used in prestressed concrete crossties is found in *Beton- und Stahlbetonbau*, V. 52, No. 7, July 1957, pp. 166-168.

Prestressed Concrete

Prestressed concrete . . . where do we stand?

ARTHUR M. JAMES, *Consulting Engineer*, V. 11, No. 4, Oct. 1958, pp. 98-102, 104, 106
Reviewed by ARON L. MIRSKY

An interesting survey of prestressed concrete's past (the human failing of overdoing in the first flush of enthusiasm over a new development is mentioned), present (the current status in codes, textbooks, materials, production facilities is discussed), and future (the need for testing facilities and model laboratories is stressed).

Latest in prestressed concrete construction (Neuere Entwicklung und Anwendung des Spannbetons mit Ausführungsbeispielen)

HANS WITTFOHT, *Beton- und Stahlbetonbau* (Berlin), V. 53, No. 4, Apr. 1958, pp. 73-85
Reviewed by VALDIS LAPSINS

Describes construction of numerous bridges, tall buildings, and underground structures which were completed in prestressed concrete.

Specifications for prestressed concrete (in Spanish)

JORGE GONZALEZ VALLENILLA, *Revista de Colegio de Ingenieros de Venezuela* (Caracas), No. 269, Aug. 1958, pp. 25-27
Reviewed by JOSEPH J. WADDELL

First the author clarifies the meaning of several Spanish language terms used to describe pretensioned and post-tensioned prestressed concrete. The balance of the article is a translation into Spanish of the specifications for prestressed concrete of the Prestressed Concrete Institute, Feb. 1, 1957.

Post-stressed concrete gasholder tank at York

T. ALBONE and H. E. MANNING, *The Structural Engineer* (London), V. 35, No. 7, July 1957, pp. 252-255
Reviewed by C. P. SIESS

Describes design and construction of circularly prestressed concrete tank 165 ft in diameter, 34 ft deep. The upper 22 ft of the wall was post-tensioned by externally applied 0.276 in. diameter wires in cables of four

wires each. Tensioning was by the Magnel-Blaton system and jacking points were 180 deg apart for a given cable.

Properties of Concrete

Elastic and permanent compression strain of concrete in rapid testing (Elastische und bleibende Zusammendrückung des Betons)

L. Bonzel, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 10, Oct. 1957, pp. 421-429
Reviewed by H. H. Werner

The author first discusses the general principles and the tests that have been carried out in this field and thence proceeds to develop a program for further tests. Equations were established for the relationship between the modulus of elasticity in compression and the compressive strengths for different conditions of storage. The tests showed that in the range of working stresses the permanent compression strains and also the compression strain at failure in the rapid test are independent of the compressive strength. To explain these phenomena, questions of pore space and the binding of water by concrete under different conditions of storage and curing are exhaustively dealt with; the relationship between the above-mentioned quantities and the changes in compressive strength and elastic properties is considered in some detail.

Freezing-thawing durability of plain concrete (Die Frostbeständigkeit von Massenbeton)

J. Orth, *Schweizer Archiv für Angewandte Wissenschaft und Technik* (Solothurn), July 1957, pp. 215-230
Reviewed by Patrick E. Murphy

In this informative article Mr. Orth continues his discussion of the factors affecting the freezing resistance of concrete. Here he treats in turn: bond between cement paste and aggregates; size and grading of aggregates; mixing and placing of concrete; and admixtures and test procedures. Each of these is discussed in detail. The effect of such variables as aggregate mineral composition, age of concrete, gap versus continuous grading, type of cement, air-entraining and antifreeze agents on concrete durability as determined by the sonic modulus is well illustrated with curves of numerous freezing-thawing tests. The freezing-thawing cycle consisted of 12 hr in air at −25 C or −30 C, and then 12 hr in water at 18 C. Prior to testing, the specimens were stored in a moist room at 18 to 20 C until they reached an age of 21 days.

Mr. Orth stresses the proper cement-water ratio and optimum amounts of admixtures. An excess of the latter can reduce the concrete durability under certain conditions. In addition, he emphasizes the importance of test materials and environment corresponding as closely as possible with the actual field conditions if laboratory research is to be useful.

General

Structure and architecture

Architectural Science, No. 2, 1957, Sydney University Extension Board, Sydney, Australia, 200 pp.

A symposium of a series of 12 lectures concerned with structure and architecture. Containing contributions from architects, structural engineers, and builders, the symposium is divided into three sections: mechanics, aesthetics, and construction.

Buildings for research

Editors of *Architectural Record*, F. W. Dodge Corp., New York, 1958, 232 pp., $9.50

A compilation from *Architectural Record* on the design of research facilities. The first section features a broad discussion of general laboratory planning and design objectives. This is followed by a section in which 44 research facilities are described in such fields as nuclear, industrial, biological, electronic, chemical, university, and military.

Many of these presentations give the reader an awareness of the problems and equipment involved in research facilities. A few presentations are so cursory that they are of little value.

Since there can be no standard model for a research laboratory—each design is dictated by the operation it houses—it is not surprising that the book does not go into the complicated details of structural, mechanical, and utility requirements and design. It is, however, a good over-all presentation and should serve as an introductory text to be read by a designer or owner before starting to plan any specific research laboratory.

DISCUSSION, PROCEEDINGS V. 55

Discussion of papers published in the July through September, 1958, JOURNALS appears in the concluding pages of this March issue, as the Institute continues its quarterly publication of discussion. Papers published October through December, 1958, will be discussed in the June 1959 issue; discussion of papers published January through March, 1959, will be published in Part 2, September 1959 ACI JOURNAL. April through June discussion will appear in Part 2, December 1959 along with index and errata for V. 55.

Disc. 55-4

Discussion of a report by ACI Committee 711:

Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)*

Discussion by Messrs. A. G. Grafflin, Angel Herrera, and the Committee will appear in the June 1959 ACI Journal.

*ACI JOURNAL, V. 30, No. 1, July 1958 (*Proceedings* V. 55) p. 83.

Disc. 55-10

Discussion of a report by ACI Committee 622:

Pressures on Formwork*

Discussion by Charles Macklin, M. R. Montgomery, Rolf Schjödt, and Wassil Weleff together with closing comments by Committee 622 will appear in the discussion section of the June 1959 Journal.

*ACI JOURNAL, V. 30, No. 2, Aug. 1958 (*Proceedings* V. 55), p. 173.

Discussion of a paper by Donald D. Meisel, Cyril D. Jensen, and Walter H. Wheeler:

Load Test on Flat Slab Floor with Embedded Steel Grillage Caps*

By G. H. BEGUIN and AUTHORS

By G. H. BEGUIN†

The authors are to be thanked for reporting their test results. Though these tests aimed mainly at acceptance of the completed structure, the data published warrant the following remarks.

Deflection—the increment due to additional loading is rather erratic between ¾A and A. It seems that the forces in the slab go from one equilibrium distribution over to another.

SR-4 gages—such strain measurement on a concrete surface is not very reliable because of the short gage length and the erratic occurrence of minute cracks over this length. In contrast to the smooth variation of the deflections, the stress increments, recorded on reinforcing steel, caused by load increments are quite irregular. The decrease noted between C and C+24 is not easy to explain.

Channel—The stress versus load diagram based on the authors' data has been drawn in Fig. A. There appears to be a definite axial force superim-

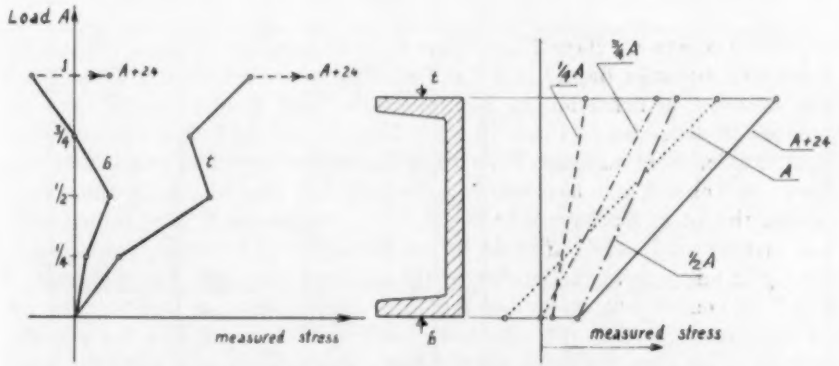


Fig. A—Stress versus load diagram for the channel, based on data by the authors

*ACI Journal, V. 30, No. 1, July 1958 (*Proceedings* V. 55). Disc. 55-6 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55).

†Member American Concrete Institute, Associate Professor of Structural Engineering, Purdue University, Lafayette, Ind.

posed on the bending moment. Data obtained at a similar structure* indicate also the presence of such a "membrane" action in the vicinity of the column. And our understanding of the structural behavior of the grillage cap would have gained somewhat if SR-4 gages had been placed on other channels.

Unhappily the authors do not mention anything about the critical points with such connections, i.e., the bond and shear stresses at the concrete-channel interface.

AUTHORS' CLOSURE

The authors appreciate Professor Beguin's discussion. He has raised some interesting questions. For clarification, Fig. B and Fig. C, stress diagrams for the 13 gage lines, are submitted.

Deflections—When Load B was added, the deflection at Point 10 doubled, and at the center of Bay 2 and Bay 4 it increased about 12 percent. However, the deflection at Point 9 decreased slightly after 24 hr. When Load C was applied the deflection at Point 10 decreased about 2½ percent and at Point 9 it increased about 53 percent. Table A shows the immediate and 24 hr decrease in maximum deflections at significant points after removal of all load. Bay 2 was under full load for approximately 2 weeks and all other bays for shorter periods.

TABLE A—PERCENT DECREASE IN MAXIMUM DEFLECTIONS

Location	Bay 2	Bay 3	Bay 6	Bay 7	Point 9	Point 10
Immediate	60	83	70	80	60	88
Recovery after 24 hr	87.8	87.7	93.5	83	79	92.7

Point 9 is between Bays 2 and 3 and midway between Columns U6 and U7. Point 10 is between Bays 2 and 7 and midway between Columns U6 and W6. The decrease in deflection at Point 9 when Load B was applied, and the decrease in deflection at Point 10 when Load C was applied, are the result of balancing loads on adjacent bays. The lag in recovery after removal of load shown in Table A was also noted in the load test that was made some years ago on the U. S. Appraisers Stores Building, Baltimore.† The reduction in unit stresses at Load C after 24 hr are the result of balancing loads and in the lag in adjustment of the slab to the changed loading. The deflection of Bay 6 at Load C plus 24 hr was 0.057 in. greater than for Bay 7. The immediate recovery of Bay 6 was much less than for Bay 7, but the recovery 24 hr after load was removed was greatest. Bay 6 bears on a 33WF141 beam at one side, whereas Bay 7 bears on a 36WF280 beam. The greater deflection of the 33-in. beam explains the greater deflection and slower recovery of Bay 6. The stiffer slab reinforcement of Bay 6 explains the high recovery 24 hr after load is removed.

Annales de l'Institut Technique du Batiment et des Travaux Publics (Paris), No. 167, Jan. 1951.
†Engineering News-Record, Oct. 18, 1934.

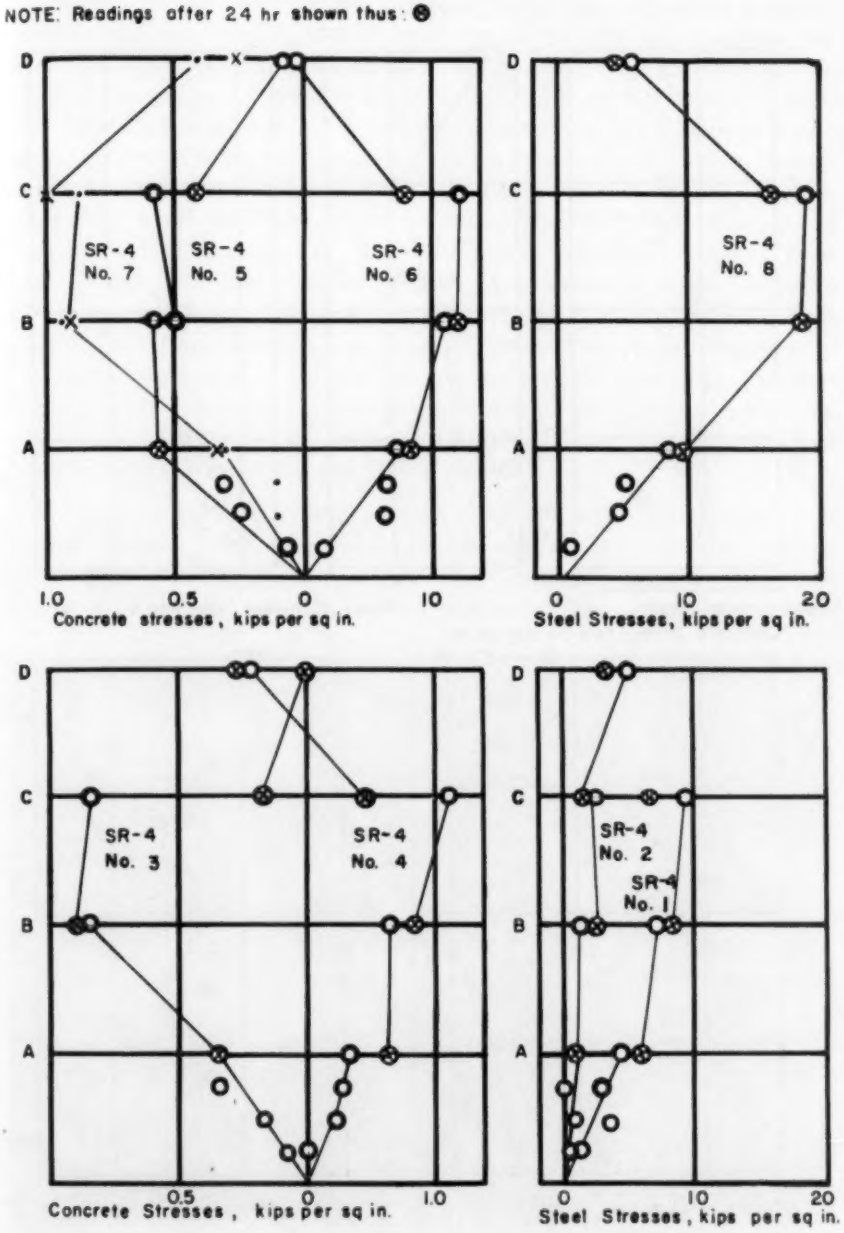


Fig. 8—Stress versus load sequence for gage lines 1-8

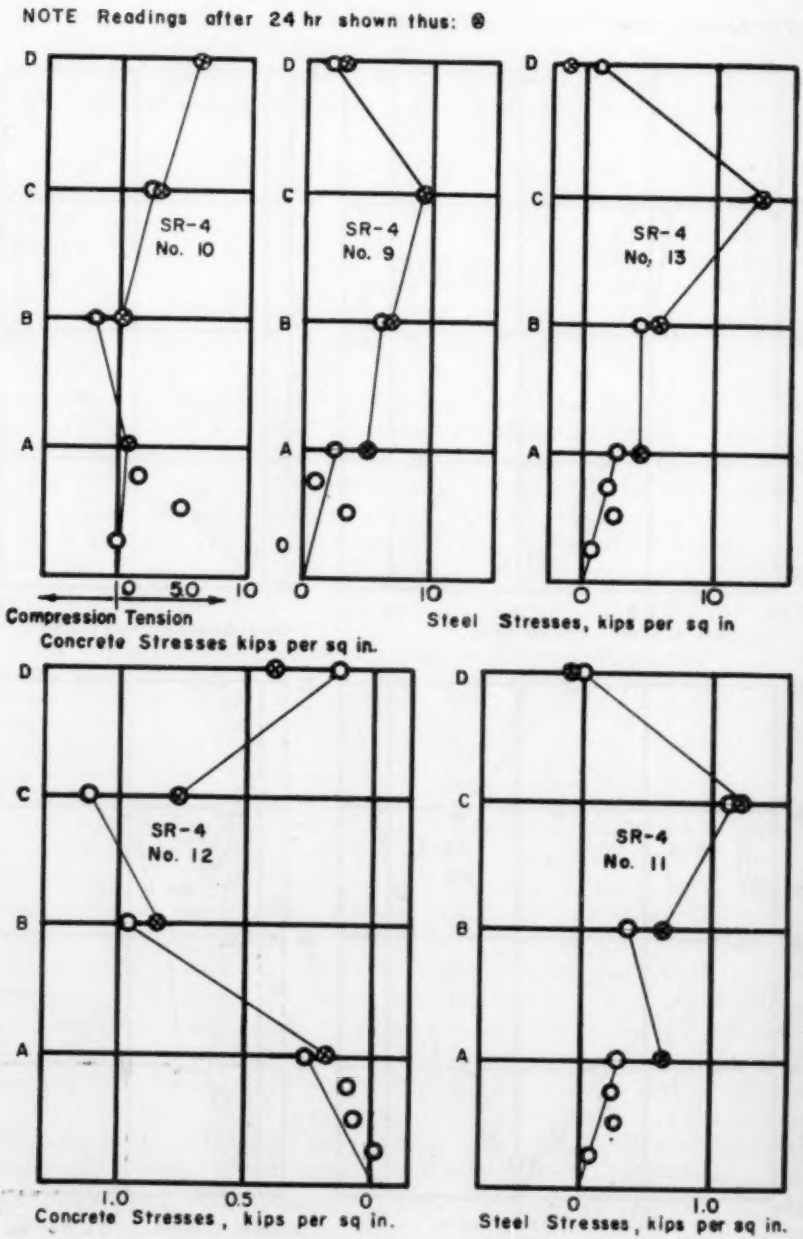


Fig. C—Stress versus Load sequence for gage lines 9-13

The reduction in the unit stress in the channel of the steel grillage cap between Load C and Load C plus 24 hr is also due to balancing loads and lag in slab adjustment to changed loading. The increase in tension in the bottom flange of the channel from 1.8 to 5.3 between Load C plus 24 hr and immediately after load is removed is due to the sudden reduction of deflection and lag in adjustment to changed loading. The readings at Point 2 on the bottom of the channel indicate slight tension at load A/4 and A/2, zero at 3A/4, compression at A, and slight tension at A plus 24. The tension increased at Load B and Load B plus 24 hr. It decreased at Load C and further at Load C plus 24 hr. With the exception of Load 3A/4, tension in the top flange of the channel increased progressively to 9.8 at Load C and reduced to 6.7 at Load C plus 24 hr. The strain gages on the channel were placed as close to the column as possible. The authors are inclined to agree with Professor Beguin that tension in the bottom flange is due to interaction of forces and suggest that because of it the neutral axis in the slab may have been lowered in the area where the strain gages were placed.

The value of the steel grillage caps as compared with standard cone caps of equal diameter was demonstrated in model tests made by the City Building Department of Minneapolis under the direction of Prof. Joseph Wise, University of Minnesota, before they were used in buildings. Fig. D shows the

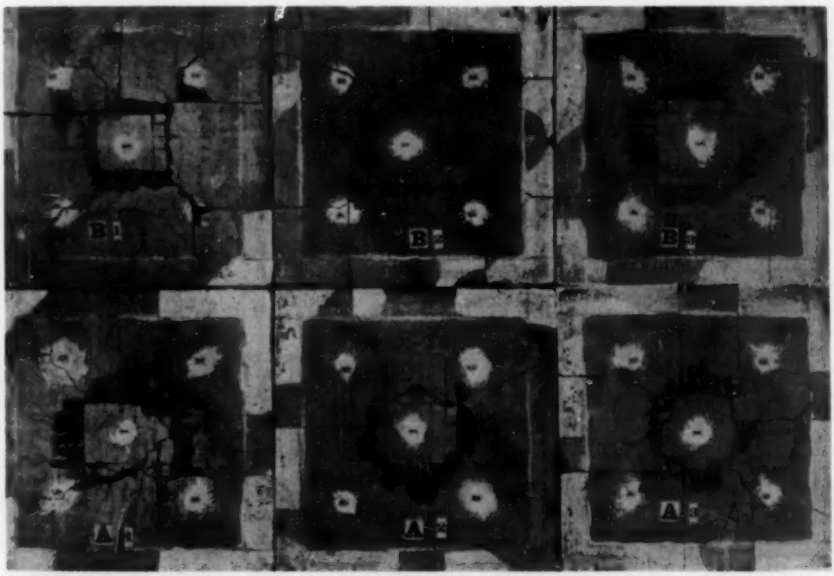


Fig. D—Scale models comparing grillage caps with standard cone caps after failure. Panels A have embedded steel grillage caps. Panels B have standard cone caps

TABLE B—MODEL TEST RESULTS

Panel	Age, days	Load at failure, lb	Panel	Age, days	Load at failure, lb
A-1	26	3000	B-1	26	3000
A-2	24	3085	B-2	24	3245
A-3	21	2700	B-3	21	2670

model slabs after failure. They were built and reinforced to scale. However the steel grillage caps were not anchored to the stub columns and the channels were light furring channels not comparable in section modulus to those normally used for buildings. Panels A have the embedded steel grillage caps. Panels B have standard cone caps. The results are shown in Table B.

Discussion of a paper by Fazlur R. Khan and Andrew J. Brown:

Load Test of 120-ft Precast, Prestressed Bridge Girder*

By M. SCHUPACK†

The authors are to be congratulated for reporting so effectively the results of this full size beam test. It is particularly gratifying to see that the structural shape tested, which is somewhat different in shape from the many tests made here and abroad on prestressed beams, behaved as calculated. This again confirms the reasonableness of our statiscal and ultimate computations for underreinforced prestressed concrete beams. This was of particular interest to me since as chief engineer of The Preload Co., Inc., at the time of design, I acted as consultant to Skidmore, Owings and Merrill in establishing various structural schemes for the highway bridges and also checked the calculations, drawings, and specifications.

The observation that "although the neutral axis at load near ultimate moves up close to the 8 in. neck, inclined tension cracks did not develop," is of particular interest. The stirrups are No. 4 at 2 ft on centers in the middle half of these beams. If we had followed the 1954 Bureau of Public Road "Criteria for Prestressed Concrete Bridges," we could have used a spacing of $\frac{3}{4}$ D or 4+ ft. It is possible that at a 4-ft spacing inclined tenson cracks might have appeared. It is our present thinking that because of the interaction of the bridge deck, diaphragms, and stringers and the resulting secondary stresses at the neck of the beam, that it is advisable to place stirrups even closer than 2 ft on centers.

It is mentioned that the beam tests were made to check workmanship and design assumptions. A beam test of an underrinforced beam of this type, in my opinion, indicates little in regard to workmanship. In composite beams, with well distributed reinforcing steel and a heavy top flange, the workmanship would have to be very poor to not achieve the load requirements as calculated. The items of honeycombing, poor grouting, or rusted tendon would generally not show up detrimentally in a beam test performed shortly after fabrication.

I believe that we have reached the stage in the prestressed concrete field, using regular aggregate, where we can be reasonably sure of the elastic and ultimate behavior of simple span structures. If the funds are available for

*ACI JOURNAL, V. 30, No. 1, July 1958 (*Proceedings* V. 55), p. 139. Disc. 55-8 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55).

†Member American Concrete Institute, Partner, Schupack and Zollman, Consulting Engineers, Stamford, Conn., and Newtown Square, Pa.; formerly Vice-President and Chief Engineer, Preload Co., Inc.

the testing of full scale members, it is suggested that some of the following information should be obtained.

1. The stresses introduced in the neck of the beam by slab wheel loads.
2. The stresses introduced in the neck of the beam in the completed bridge due to the interaction of slab, diaphragms, and stringers.
3. Visual observation of the effectiveness of the grouting at various points along the beam.
4. Long term observation of deflections to better understand camber problems.
5. Shear tests.

Discussion of a report by ACI Committee 612:

Curing Concrete*

By J. E. JELLICK and COMMITTEE

By J. E. JELLICK†

This discussion comments on the following passage from the paragraph on optimum practice for curing horizontal units (p. 166). "*Initial Curing*—Immediately after the finishing operations are completed, the concrete should be covered with two thicknesses of an approved woven fabric, a quilted fiber mat, or other covering of an approved absorptive material thoroughly saturated with water when placed."

At certain times of the year, particularly during summer, when low humidity and drying winds prevail, we have had considerable shrinkage cracks appear in flat slabs, especially when said slabs were on a non-absorbent base, such as on a concrete slab in casting concrete panels for tilt-up construction.

When a steel troweled surface is required, it is impossible to place any woven fabric, quilted fiber mat, or other covering of an approved absorptive material immediately after the final troweling without marring the surface. Therefore, with the necessary delay in applying a desirable covering, too much water has evaporated from the surface, with the result that shrinkage cracks appear, either soon after or by the end of the first day. Regardless of the brand of cement, or the brand of curing compound applied for curing, or whether a retarder was used or not, we have experienced these shrinkage cracks regularly during the above mentioned conditions.

Following the practice of the concrete irrigation pipe manufacturers we have adopted the use of a special nozzle which produces a fog spray, as the name implies. With this special fog nozzle we are able to apply a fog spray of water *immediately after the final troweling*. This is continued several times during the first day. On the following morning any of the standard brands of curing compounds may be applied, or any other conventional curing methods such as mentioned above.

This method is so effective and so inexpensive, compared to many other methods which have been publicized, that it should not be overlooked. It has been adopted by many architects, engineers, and contractors in northern California.

*ACI Journal, V. 30, No. 2, Aug. 1958 (*Proceedings* V. 55), p. 161. Disc. 55-9 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55).

†Member American Concrete Institute, Manager, Portland Cement Information Bureau, San Francisco, Calif.

COMMITTEE CLOSURE

The committee agrees with Mr. Jellick that a fog spray can sometimes be used to advantage in the early stage of curing horizontal units to control plastic shrinkage cracking. The problem and various corrective measures, including fog spray, are discussed in considerable detail by Lerch.*

It should be noted that the time of starting fog spraying can be critical. If started too soon, it may result in "washing" of the newly troweled surface. If deferred too long shrinkage cracks will already have formed. It is important that the spray is actually a fog-spray, not a jet-spray, and that dribbling of water on the surface is avoided. In much flat work, of course, slight marring of the surface by wetted coverings would not be considered detrimental.

The committee has reconsidered one statement in the report, on p. 169, as follows: "In any event, the internal concrete temperature during hydration should not reach a value in excess of 20 F above the mean annual temperature." As desirable as this may be, it is recognized that often it would be impractical to attain, especially if pipe-cooling after placement is not provided. However, it remains important to keep the temperature of mass concrete as low as practicable, and it is proposed that the statement be reworded as follows: "In any event, the concrete placing temperature should not reach a value in excess of 20 F above the mean annual temperature."

The committee has received a number of inquiries and suggestions concerning the report, though not in the form of formal discussion, which will be helpful in preparing an ACI Standard on the recommended practice for curing concrete. The committee will welcome additional constructive criticism as an aid in this preparation.

*Lerch, William, "Plastic Shrinkage," ACI JOURNAL, V. 28, No. 8, Feb. 1957 (*Proceedings* V. 53), pp. 797-802; discussion, V. 29, No. 6, Part 2, Dec. 1957 (*Proceedings* V. 54), pp. 1341-1346.

Discussion of a paper by Tien S. Chang and Clyde E. Kesler:

Fatigue Behavior of Reinforced Concrete Beams*

By R. TAYLOR and AUTHORS

By R. TAYLOR†

Few experimental data exist on the effect of repeated loading on the shear strength of reinforced concrete beams so that the investigations reported by the authors, together with that reported by Stelson and Cernica,[3] form an important contribution to the knowledge of the strength of concrete members. As part of a comprehensive program on the shear strength of reinforced concrete beams being undertaken at the Building Research Station, England, repeated loading tests have recently been carried out on beams without shear reinforcement.

REPEATED LOADING TEST PROGRAM

Specimens

The main details of the beams are summarized in Table A. All the beams were 7½ x 10 in. in cross section, 7 ft 6 in. long. Two groups of beams were tested—one reinforced with plain round mild steel bars (yield stress: 43,000 psi), and one incorporating cold-worked deformed bars having the herringbone pattern of deformations (0.25 percent proof stress, 66,000 psi). In each group, beams with two different percentages of reinforcement were tested, there being 4 beams of each percentage of plain bars and 3 beams of each percentage of deformed bars. In the beams with the deformed bars all the reinforcement was straight, the bars extending to the ends of the beam. Of each set of four beams reinforced with plain bars, two were reinforced with straight bars and two with bars having hooks at the ends.

The quality of the concrete was maintained constant throughout the series. For cubes stored in water and tested at 34 days (24 hr after commencement of the pulsating test) the mean of the average crushing strengths was 4280 psi with a coefficient of variation of 4.6 percent.

Tests

The beams were supported over a 6-ft span, using rubber strip and steel roller supports to obtain stability and also to insure that no external longitudinal forces occurred on the beams during the tests. A single point load was applied at the midspan with a 25-ton hydraulic jack connected to an Amsler pulsator operated at 250 cycles per min.

*ACI JOURNAL, V. 30, No. 2, Aug. 1958 (*Proceedings* V. 55), p. 245. Disc. 55-14 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55).

†Senior Scientific Officer, Building Research Station, Department of Scientific and Industrial Research, Garston Watford, Herts., England.

TABLE A—RESULTS OF TESTS

Reinforcement			Maximum of load range		Number of repetitions to:		
	Area						
Bars*	Sq in.	Percent	Tons	Percent†	Diagonal cracking	Collapse	End of Test
4 ½-in. deformed	0.785	1.21	6.0	70	—	—	1,483,000
			6.25	73	700,000	1,340,000	
			6.5	76	40,000	40,000	
5 ½-in. deformed	0.981	1.49	6.5	70	—	—	3,000,000
			7.0	76	126,000	126,000	
			7.0	76	35,000	35,000	
(Hooked)			6.75	74	—	—	1,467,000
			7.0	77	31,000	—	1,400,000
6 ½-in. plain (Hooked)	1.177	1.83	7.0	77	31,000	121,000	
			7.25	80	3600	3600	
(Hooked)			6.5	65	—	—	1,300,000
			7.25	72.5	31,000	—	1,647,000
5 ⅝-in. plain (Hooked)	1.534	2.33	7.5	75	222,000	222,000	
			7.75	77.5	2600	2600	

*The effective depth of the reinforcement was 8.75 in.
†The maximum of the load range expressed as a percentage of the static diagonal cracking load.

The minimum of the applied load range was approximately 17 percent of the average diagonal cracking load,* determined in static tests of two or three similar beams which had been previously tested. During the test on a particular beam the load range was kept constant, but the maximum of the range was varied throughout the series in an attempt to determine the maximum load that could be applied under such conditions without causing the formation of diagonal cracks. The pulsating load was applied until the collapse of the beam or, in the event of the load not causing collapse, until approximately 1,500,000 repetitions had occurred, although in the case of one beam 3,000,000 pulsations were applied.

Test results

The results of the tests on the beams are summarized in Table A. The maxima of the load range are noted, and these are also expressed as a percentage of the diagonal cracking load in the static tests. While the tests were necessarily limited in number and scope the results appear to indicate that:

1. The relationship between the shear strength of beams under repeated loading and that under static loading conditions is similar whether the beams are reinforced with cold-worked deformed bars or with plain mild steel bars.

2. For the conditions of the test, i.e., a range of loading applied to a beam at a rate of 250 cycles per min with a minimum of the range at approximately 17 percent of the static diagonal cracking load, the maximum of the range which could be sustained by the beam for 1,000,000 repetitions without causing diagonal cracking is approximately 70 percent of the static diagonal cracking load. When this load is slightly exceeded collapse may occur following the gradual formation of a diagonal crack, but when the maximum of the range exceeds approximately 77 percent of the static diagonal cracking load, formation of the diagonal crack and collapse may be very rapid.

*The load at which a major inclined crack formed.

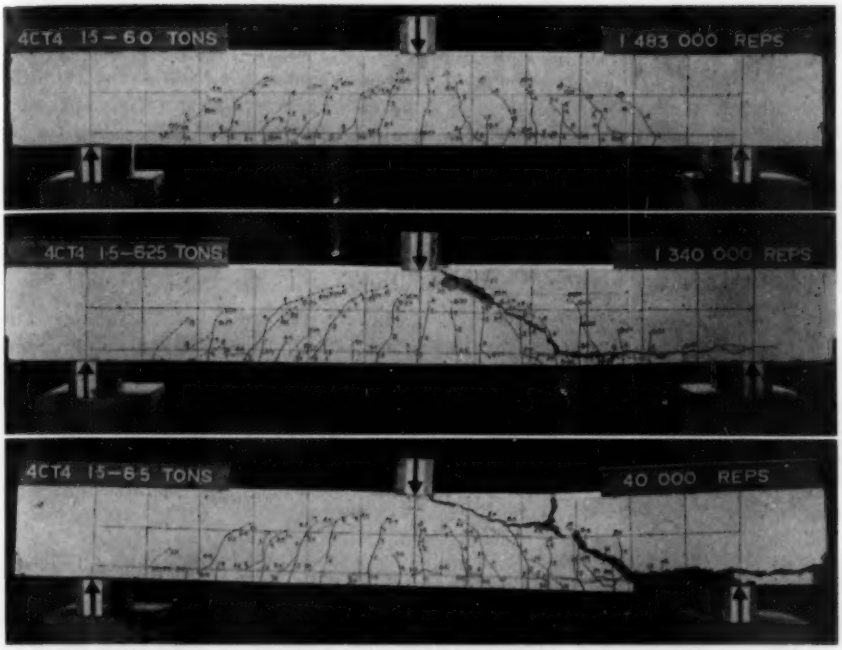


Fig. A—Crack patterns after repeated loading tests on beams with four ½ in. diameter cold-worked deformed bars

3. In beams so loaded that a/d is comparatively large (in these tests this ratio was 4.1) anchorage failure of straight, plain round bars having a rusty surface is unlikely to occur except as a secondary phenomenon of the collapse mechanism.

COMPARISON WITH PREVIOUS TESTS

This work is similar to that previously reported by the authors[1] and it is interesting to compare results. This comparison is limited to the phenomenon of diagonal cracking since it is considered that this stage and not actual collapse must, for practical purposes, determine the failure criterion of beams without shear reinforcement. The reasons for this are:

1. When a beam is loaded in such a way that the propagation of the diagonal crack is not inhibited in any way by the compressive stresses set up by the load, e.g., by loading through secondary beams framing into the sides of the beam, then the beam collapses at the formation of the diagonal crack (*cf.* Ferguson[4]).

2. With beams loaded on the top surface, collapse of the beam often occurs immediately at the formation of the diagonal crack, when a/d is relatively large.

3. When a/d is relatively small, beams loaded on the top surface may sustain loads higher than that causing diagonal cracking but only after considerable widening of the inclined cracks.

In several respects the results of the two investigations are similar, as, for example, in the behavior of the beams under the pulsating load with comparable modes of collapse (see Fig. A). In addition in the tests at the Building Research Station it was observed that in the beams in which a diagonal crack formed it did so within the first few hundred thousand repetitions of load—the maximum number of repetitions to form a diagonal crack was 700,000. This was generally the case in the tests by the authors although in three of the 39 beams tested the number of repetitions to cause the formation of a diagonal crack was approximately 1,000,000, and in one beam a diagonal crack formed after over 6,000,000 repetitions.

In one important respect, however, the results are quite different. The Building Research Station's results indicate that, for 1,000,000 repetitions, the fatigue limit would be approximately 70 percent of the static diagonal cracking load while the authors' results indicate that the fatigue limit giving a low probability of failure for the same number of repetitions would be approximately 53 percent. Such a large difference in results makes it necessary to study the conditions of test in the two series in an attempt to explain the difference.

The rate of loading used by the authors was 440 cycles per min, compared with 250 cycles at the Building Research Station. Since previous tests[2] on plain concrete beams showed that the bending tensile strength is unaffected by the frequency of the pulsations over the range 70 to 440 cycles per min, it would seem unlikely that this difference in the tests contributed to the difference in results.

The size of beam tested by the authors was 4 x 6 in. with 1 in. maximum size aggregate (compared with Building Research Stations beams, 7½ x 10 in, with ¾ in. maximum size aggregate). In their static tests on similar beams the authors had found a considerable increase in shear strength in terms of nominal shear stress as compared with larger beams tested by previous investigators. It was concluded that this was due to the difference in the dimensions of the beams and, by dimensional analysis, an expression including the "size effect" was derived for the diagonal cracking load. The difference in static diagonal cracking strength due to the size effect for the beams used in the two investigations is, according to this expression, approximately 12 percent. It is possible that this size effect is reduced under repeated loading. If this were so it would account for part of the difference in the relationships between the diagonal cracking load under static loading conditions and under repeated loading conditions.

It might also be pointed out that the nomograph of the expression* does not predict accurately the results of the static tests reported by Stelson and Cernica which were also carried out on small beams (5 x 5½ in. using

*There appears to be some discrepancy between the nomograph and the expression as printed. The nomograph has been used for this comparison since it agrees more closely with the tabulated data in the paper than does the expression. Curiously, however, the expression predicts the results of Stelson and Cernica more accurately.

½ in. maximum size aggregate). The nominal shear stress at diagonal cracking predicted for these beams is approximately 12 percent above the average value of several tests. While it is realized that the expression was derived from limited data as regards the effect of small beams, it is the opinion of the writer that, if it is desired to include a size effect, consideration should also be given to the maximum size of the aggregate used. (Compare the different ratios of effective depth:maximum aggregate size used in the three investigations.)

The minimum of the load range used in the tests by the authors was small, approximately 2 percent of the static diagonal cracking load (*cf.* BRS 17 percent). It has been well established by investigators that the fatigue strength of a structural member is dependent not only on the maximum load applied to the member, but also on the range of load applied. The modified Goodman diagram for tests on plain concrete beams reported elsewhere[5] in the fatigue symposium indicates that, in the range where there is no stress reversal, an increase in the minimum of the applied load corresponding to 15 percent of the static flexural strength would result in an increase in the maximum of the range of about half this amount. It is likely that a similar difference has occurred in the results of the two investigations due to the difference in the range of applied load.

The range of loading adopted by the authors does, of course, give the worst effect that could possibly occur. Usually, however, the dead load carried by a reinforced concrete member is a considerable proportion of the working load. Since it is only live loading which makes it necessary to consider fatigue resistance, it appears more realistic to conduct tests in which a range of pulsating load is superimposed on a load representing a condition of dead load. In the case of the Building Research Station tests, the minimum load of 17 percent of the static diagonal cracking load would be equivalent to between 33 and 50 percent of the working load depending on what load factor from 2 to 3 is adopted for determining the working load.

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4. Ferguson, Phil M., "Some Implications of Recent Diagonal Tension Tests," ACI JOURNAL, V. 28, No. 2, Aug. 1956 (*Proceedings* V. 53), pp. 157-172.

5. Murdock, John W., and Kesler, Clyde E., "Effect of Range of Stress on Fatigue Strength of Plain Concrete Beams," ACI JOURNAL, V. 30, No. 2, Aug. 1958 (*Proceedings* V. 55), pp. 221-231.

AUTHORS' CLOSURE

Mr. Taylor has made a substantial contribution in the data and discussion he has presented.

The results of Stelson's and Cernica's tests[3] differed approximately 12 percent from that predicted by the nomograph,[1] as indicated by Mr. Taylor,

however, this variation is not unreasonable for reinforced concrete beams. The amount of data available for small beams is not great and variables, such as aggregate size should also be considered, in addition to beam size. However, sufficient data for this purpose are not available for all the existing tested beams.

While the cracking load must necessarily be considered as most important in design, the factor of safety used might well depend on whether the beam would collapse at the cracking load or carrying additional load. Hence failure loads should not be eliminated from study.

Mr. Taylor indicates the importance of range of stress in reinforced concrete beams. The difference in the minimum load between his and the authors' tests, amounting to 15 percent of the ultimate strength, should and did make considerable difference in the results obtained. More data on this variable is needed for reinforced concrete beams and as pointed out by Mr. Taylor the dead load on a member in service may be a considerable percentage of the total dead and live load for the member.

Discussion of a paper by Frederick P. Wiesinger:

Design of Symmetrical Columns with Small Eccentricities in One or Two Directions*

By GEORGE B. BEGG, JR. and ORLEY O. PHILLIPS and MORTON SOLOMON
By GEORGE B. BEGG, JR.†

Professor Wiesinger has introduced an extensive design method for columns subject to axial and bending load within the limitation of $e/t \leqq 2/3$. He covers a wide range of column shapes under "shape factors," α_g, and an equally good range of reinforcing patterns summed up in "pattern factors," α_s [See his Eq.(1) and (2)]. Using these new dimensionless variables, he rephrases

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leqq 1‡$$

to a new equation

$$A_g = \frac{N}{Q\,[0.225 f_c + f_s\, p_g]} + \frac{Ne/t}{F_b\,[\alpha_s + \alpha_s\,(n-1) g^2\, p_g]}$$

This new equation is then scrutinized and attention is immediately concentrated on the denominators of the two terms (the numerator terms are explicit in the given design data). Tables are presented for these denominator terms and essentially are built aroung p_g and the shape and pattern factors. The method presupposes a trial and error design with rapid convergence growing out of use of the tables. The nature of this convergence is illustrated in examples and the construction of a chart that pivots on assuming a column size. I would like to comment on this subject and suggest criteria for evaluating new and current methods of design with special emphasis on columns within building frames.

Interest in the convergence indicated by the author has been widespread for the last 30 years. The Public Buildings Service, for example, has been preparing tables and straight line and nomographic charts since the early 1930's, aiming at meeting our limitations in time, accuracy, and personnel in large scale designing of concrete columns for load and moment.

This interest has been wider in scope than that permitted by limitation of this paper, i.e., $e/t \leqq 2/3$. However, although any tables or charts dealing with combined axial and bending load will only be complete when the

*ACI JOURNAL, V. 30, No. 2, Aug. 1958 (*Proceedings* V. 55), p. 273. Disc. 55-17 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55).

†Member American Concrete Institute, Supervisory Structural Engineer, Public Buildings Service, Washington, D. C.

‡Eq. (18) Section 1109(a) of ACI 318-56.

cracked-section area, i.e., $e/t \geqq 2/3$ is covered, the limitation of $e/t \leqq 2/3$ is not highly restrictive. Thus in a building frame if the exterior roof columns, where the axial-bending load ratio can be very low, are successfully handled, we can hope to design most of the remaining columns within our scope.

Now, accepting the interest in and the adequate range of scope permitted by our limitation, is the convergency given in Professor Wiesinger's paper the best approach to the problem? The answer depends on what the designer conceives the design question to be and what then to emphasize in speed, new terms, wide range, compact presentation, etc. in answering the question. The author has certainly developed a compact presentation with wide range. More of this later.

TIED COLUMN CHARTS

I will present here some examples of tied column charts that concentrate on the question that confronts the designer of a multistory concrete building as he comes down his column run-down format. On any given floor the designer has an accumulated load and a bending moment at his column. What now to do with them?

The design charts have the axial load and moment as coordinates. Concrete and steel stresses, and column size are held constant for a given chart. The charts present a family of straight lines that give the number and size of reinforcing bars. The straight line plot reflects a solution of the combined stress equation.

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leqq 1.0$$

The upper limit of the charts is governed by Section 1104(b), ACI 318-56, where only 4 percent p_g is permitted to withstand the axial load alone. This requirement is shown as a horizontal line on the charts and is called simply "cutoff load." Below this load advantage is taken of the same section of the code that permits up to 8 percent for combined action. The lower cutoff line is the $e/t = 2/3$ requirement. As an aid to the charts and for preliminary design, indexes precede the charts (Fig. A and B).

Notation

A_s = effective cross sectional area of vertical column bars in compression, sq in.

e = eccentricity of the resultant load on a column, measured from the gravity axis, in.

f_c' = compressive strength of concrete at 28 days, kips per sq in.

f_s = nominal allowable stress in vertical column bars, kips per sq in.

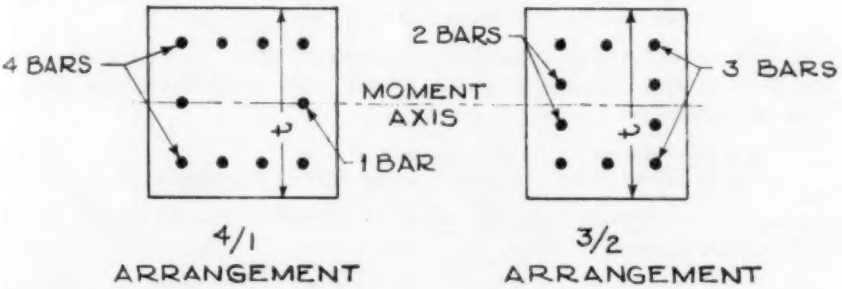
M = bending moment applied to reinforced concrete column, ft-kips

N = axial load applied to reinforced concrete column, kips

t = over-all depth of square or rectangular column section, in.

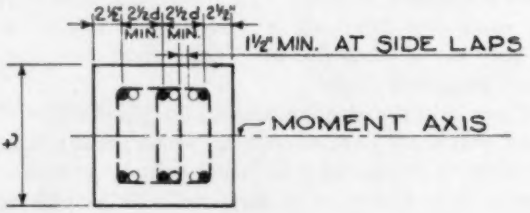
Fraction following reinforcement size, e.g., 4/1, 3/2, etc., indicates position of vertical column bars as follows: Numerator indicates the number of bars in the face normal to the direction of moment. The denominator indicates the number of intermediate bars in the other face. Thus, Arrangements 4/1, and 3/2 are as shown below. This notation applies to four-face A_s charts.

No fraction following reinforcing size indicates all reinforcing is divided equally between two opposite faces (two-face A_s on charts).



Limitations

1. When the intersection of any load and moment value does not fall on a line in the indexes for column size or in the charts for vertical column bars, any line above or to the right of the point is safe.
2. The vertical column bars shown on charts are placed in one vertical plane per column face.
3. For lateral ties requirement see Section 1104(c), ACI 318-56.
4. On the charts where a solid reinforcing bar line discontinues, a straightedge placed along the line can be used to establish the full range of the line. Indications in parentheses represent alternate bar combinations. Where the alternates are no longer identical the range is indicated by a dashed line.
5. Some reinforcing patterns shown in square column charts presuppose butt welding at splices. Therefore, patterns should be reviewed for spacing if side lapping of reinforcing is used.
6. All rectangular column charts are limited to case of bending in direction of strong axis.
7. All reinforcing patterns shown in rectangular columns are for side lapped bar spacing.
8. Distance from center of vertical column bar to face of column in all cases is 2½ in. For general spacing of vertical column bars and spacing of side lapped splices, see figure below.



Example of chart use

Given the design load and moment, stress criteria, and the charts, and keeping in mind any architectural limitation, economic limitation (maximum p_g permitted by competition), etc., the designer goes to the index (Fig. A

and B) and picks a column size. Turning to the charts* having the column size given by the index, he picks the reinforcement and writes it into his column schedule. For example:

Given: $M = 30$ ft-kips, $N = 210$ kips, $f_c' = 3.0$ kips per sq in., and $f_s = 20.0$ kips per sq in.
Find: Minimum square column size and reinforcement
Solution: Entering square column index (Fig. A) with criteria, pick a 16-in. square column. Turn to the 16-in. square chart (Fig. C) and using same criteria pick six #11 bars. Therefore column is 16 in. square with six #11 bars.

Now if architectural consideration would not permit a 16-in. face normal to the plane of bending, the following column sizes could have been picked from the indexes as:

For $f_c' = 3000$ psi	For $f_c' = 3750$ psi
Use 12 x 20-in. column	Use 14-in. square or 12 x 18-in. columns (Fig. D)

Reinforcement for these columns would be picked in manner similar to the 16-in. square column.

ADVANTAGES AND DISADVANTAGES OF THE CHARTS

In the method I propose, the designer picks his column by inspection of the indexes and charts once he has carried his column run-down format to an accumulated load and bending moment. There is no need to assume column size, p_g, or to compute shape and pattern factors. The convergence is immediate and no new terms beyond load and moment are introduced. Professor Wiesinger depends on new terms, assumptions, computations, and finally a trial and error convergence.

However, I have covered only a small part of the range included in his method; for example, I have not included in this paper spirally reinforced columns, reinforcing bars with $f_s = 16$ kips per sq in. etc. Yet there are many charts involved, and the full range will be bulky. The computing and plotting of numerous charts is time consuming in original work and in checking. Finally, in spite of good indexes the designer might never have the feeling of familiarity with a thick book of charts that he would have with the author's compact method. To these objections I reply that once the charts are completed, the bulk of presentation will in no way diminish the speed and easy use of these charts. The method used is, finally, a matter of the individual designer's choice.

There are a few additional advantages and possibilities of these charts. Most tables and charts are built around p_g, which greatly simplifies the computations and plotting compared with using actual bar make-up. However, after convergence, it is necessary in such methods to pick actual bar combinations to meet this p_g; since the steel area chosen is only accidentally the area required, the final load-moment carrying ability of the column is seldom

*Only two typical charts are included with this paper. A complete set (69) of the available design charts may be ordered from ACI headquarters for $6.00 (approximate cost of reproduction). The 8½ x 11-in. charts cover square tied columns, ranging from 12 x 12 in. to 24 x 24 in. in 2-in. increments, for $f_c' = 3000$ psi and 3750 psi, and with reinforcement in either two faces or four faces. The charts for rectangular tied columns with reinforcement in four faces, with $f_c' = 3000$ psi and 3750 psi, cover sizes ranging from 12 x 14 in. to 12 x 34 in. in 2-in. increments.

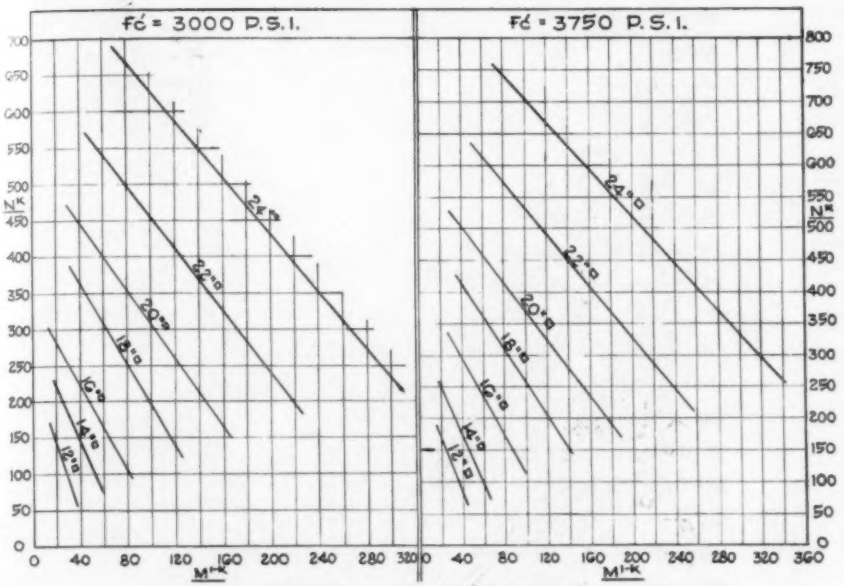


Fig. A—Index of square tied columns

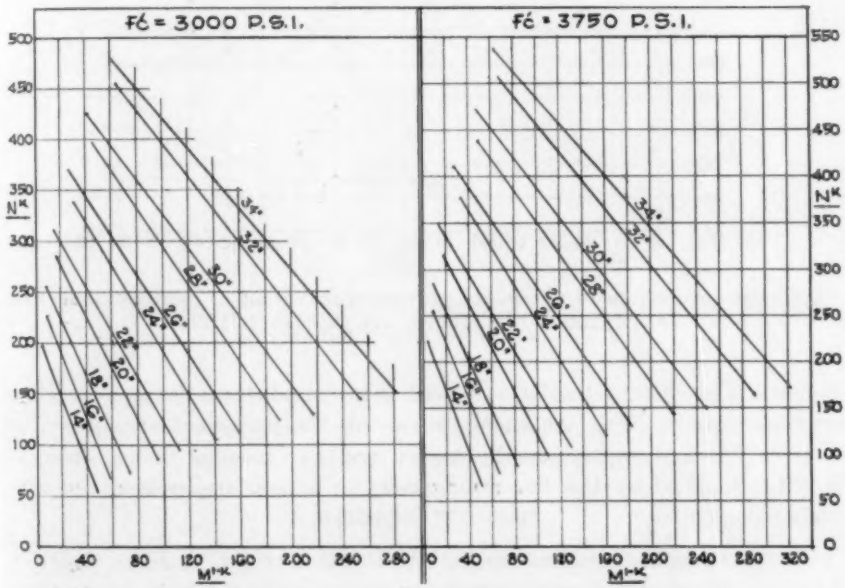


Fig. B—Index of 12-in., rectangular tied columns

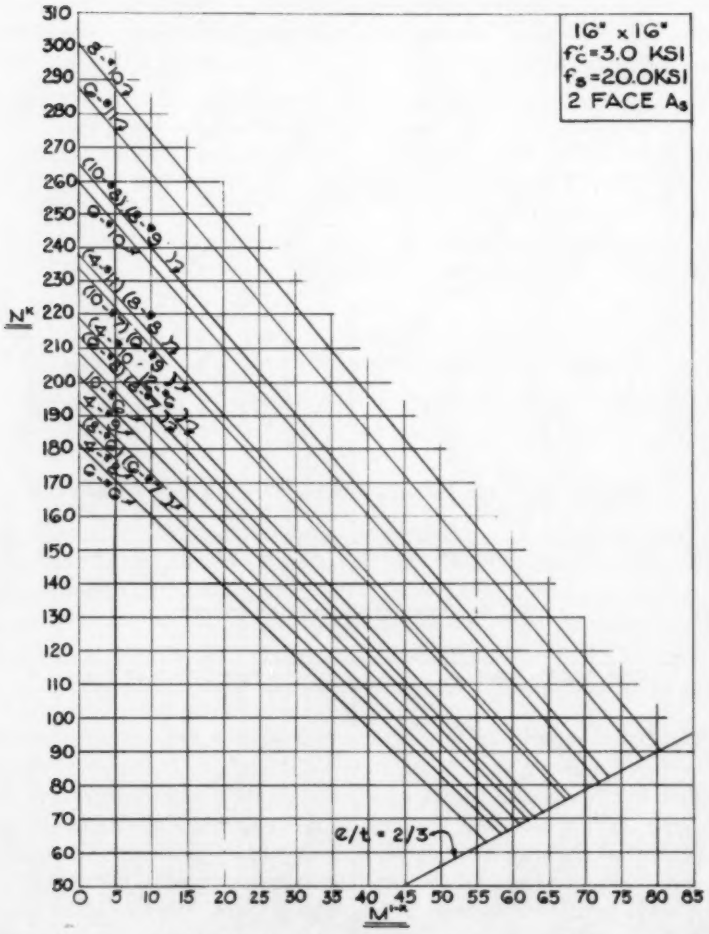


Fig. C—Design chart for 16 x 16-in. column, typical of a group of charts available from ACI at cost of reproduction (see footnote, p. 1020)

known. Therefore, if the load-moment is increased, the column must often be recomputed. With the charts presented, the load-moment capacity of a given bar make-up is visually known and any increase in load-moment is either handled by this bar arrangement or a new arrangement immediately indicated.

On the subject of moment on both principal axes, i.e., $f_a/F_a + f_{bx}/F_b + f_{by}/F_b = 1.0$*, evidence seems to point to the indexes of these charts as a point

*Eq. (19) Section 1109(a) of ACI 318-56.

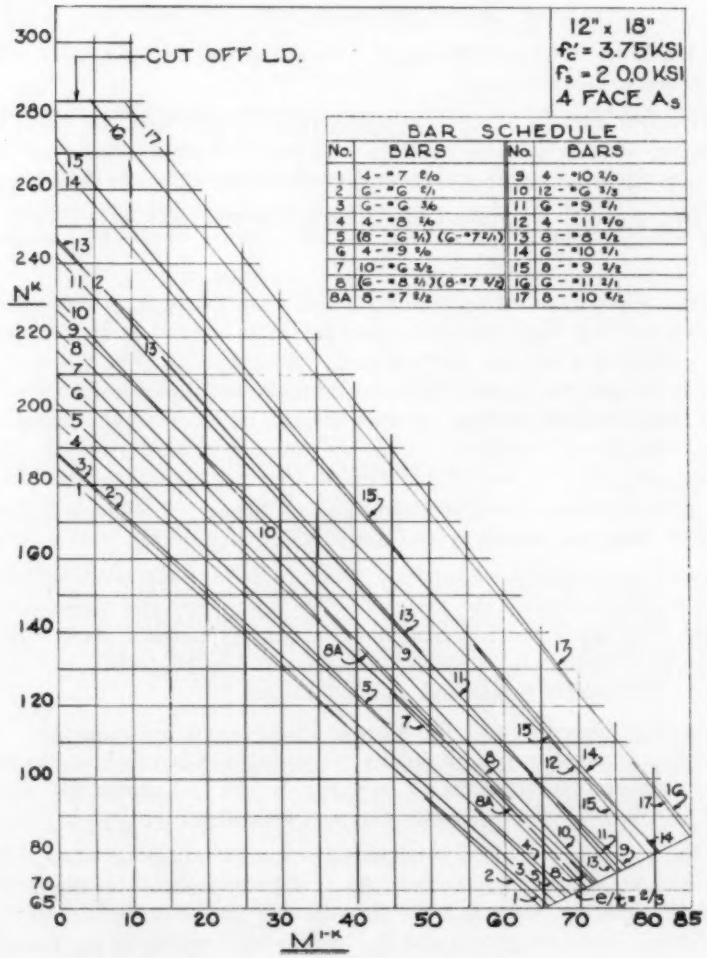


Fig. D—Design chart for 12 x 18-in. column, typical of charts available from ACI at cost of reproduction (see footnote, p. 1020)

of departure for this problem. As far as we have gone on our charts, we find that the relative insensitivity of the indexes suggests that some modified superposition of moments will lead to a fairly rapid convergence.

ACKNOWLEDGMENT

I would like to thank Messrs. J. W. Dunham, H. M. Zimmerman, G. E. Hartshorn, and V. R. Rice of the Public Buildings Service for considerable help and encouragement on the continuing work on these charts.

By ORLEY O. PHILLIPS* and MORTON SOLOMON†

The paper by Professor Wiesinger is excellent and will provide a valuable aid in concrete column design.

It would be well to bring to the reader's attention that the value of Ne' may be replaced by a value of M in in.-kips. Column moments are usually found directly from moment distribution sheets on which the building frame is analyzed. If the M value is exceedingly large, e' must be found to determine if the e'/t value is not more than 2/3 in either direction as allowed by code.

Tables 3 and 4 have values tabulated with a maximum $p_g = 0.04$. Spirally reinforced columns may have a maximum $p_g = 0.08$ as may tied columns with a combined axial and bending load. Section 1104(b) of the ACI Code notes the limitations for tied columns. An extension of these tables would allow a slightly smaller column section with an increase in reinforcing.

AUTHOR'S CLOSURE

The author appreciates the discussions of Messrs. Phillips and Solomon and of Mr. Begg, as well as several direct communications.

Outstanding among the latter is a letter from Mr. Müller (who has published a paper bearing on the subject‡). He raises the valid question of how to simplify the design of columns that are not symmetrical about two principal axes; unfortunately, it seems that it is not possible to boil this problem down to a few simple tables and design formulas.

Concerning the suggestion by Messrs. Phillips and Solomon that Tables 3 and 4 be extended to include higher percentages of reinforcing, the author feels that this can be accomplished properly by ACI Committee 317 when the *Reinforced Concrete Design Handbook* is next revised.

Mr. Begg's charts cover an admittedly narrow range of cases. About one-third of the charts duplicate parts of the information available in the *Concrete Reinforcing Steel Institute Handbook*. The rest presents "4 Face A_s" patterns, which might be of some use for high values of p_g, though uneconomical to resist bending. The use of these patterns for low values of p_g can hardly be justified.

The author's purpose in developing the method presented in the paper was not so much to deal with the somewhat trivial case of bending about one axis, but to provide a method for bending about both axes. It is hoped that the method presented will be of value to the practicing designer.

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‡Muller, L. S., "Eccentrically Loaded Corner Columns (LR 47-75)," ACI JOURNAL, V. 22, No. 7, Mar. 1951 (*Proceedings* V. 47), pp. 562-565.

Discussion of a report by Subcommittee on Proportioning Lightweight Aggregate Concrete, ACI Committee 613:

Proposed Recommended Practice for Selecting Proportions for Structural Lightweight Concrete*

By E. L. HOWARD and W. E. MOULTON, and SUBCOMMITTEE

By E. L. HOWARD and W. E. MOULTON†

The subcommittee defines lightweight concrete by referring to 115 lb per cu ft *air dry*. This condition is not definite enough for specifications.

The subcommittee barely mentions ACI Standard "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)." The subcommittee suggests that these standards may be used with certain aggregates. More emphasis should be placed on the use of the standard practice for proportioning.

Our use of the standard practice (ACI 613-54) with three types of aggregates has produced excellent concrete. All aggregates are used in saturated condition. Pumice aggregate is stored in ponds and bunkered immediately before batching. The expanded shale, crushed and uncrushed, is saturated in the batch plant bunkers before use.

Free moisture of fine aggregate may be determined with moisture meters and samples of the coarse aggregate can be quickly surface dried and weighed. We can and do control the water-cement ratio and can predict the compressive strengths as well as with conventional aggregate.

Coarse aggregate gradings are no problem for us. Our local producers supply the material in two sizes—1 in. to ⅜ in. and ⅜ in. to No. 4 mesh. We use as little fine lightweight aggregate and as much natural sand as possible and conform to specified weight. Variances in the fines are a problem in proportion to the amount used. For design purposes we use a fineness modulus of 3.00.

We agree with the importance placed on the use of air. The pressure type meter is successfully used when the aggregates are saturated before use.

If the involved methods of estimating proportions as outlined by the subcommittee in Chapters 3 and 4 are indeed desirable for aggregates familiar to the subcommittee, then we in California are a blessed people.

We can and do proportion good workable mixes based on absolute volumes of saturated surface dry materials. Field control must solve two problems:

*ACI JOURNAL, V. 30, No. 3, Sept. 1958 (*Proceedings* V. 55), p. 305. Disc. 55-18 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55).

†Members American Concrete Institute, Chief Testing Engineer and Testing Engineer, respectively, Pacific Cement and Aggregates, Inc., San Francisco, Calif.

(1) proper aggregate saturation and (2) changing specific gravity. The first problem is solved when the batch plant personnel use adequate care in handling materials. The second problem will be solved only when the aggregate producers furnish aggregate of controlled density. Yield determinations on the job with batch adjustments are the stop gap methods suggested by the committee and practiced by us all.

We have not found air content change a problem as suggested by the subcommittee's reference to unit weight changes in job control. We believe the measured air will change greatly with the degree of aggregate saturation. The actual air entrained is no more difficult to control than in conventional concrete.

The proposed recommended practice is not useful to us except as it refers to the existing standard, ACI 613-54, and cautions the user of lightweight concrete of the pitfalls peculiar to this type of material.

SUBCOMMITTEE CLOSURE

The subcommittee was pleased to read that Messrs. Howard and Moulton are continuing to produce satisfactory lightweight aggregate concrete as they described in the ACI JOURNAL of June, 1955, p. 1061, and that they have experienced no difficulty with the absolute volume method of mix proportioning (ACI 613-54). However, for reasons clearly stated in the subcommittee report, most producers of lightweight aggregate concrete have not been able to proportion and control mixes adequately by use of the water-cement ratio and absolute volume method.

Messrs. Howard and Moulton stated that appropriate aggregate saturation is a field control problem. This is emphasized in the subcommittee report and is in part the reason for the particular method of proportioning suggested. The success obtained by the discussers probably lies in complete saturation of the aggregates prior to batching. However, saturation of the aggregate is undesirable in areas where the concrete may be subjected to freezing and thawing exposure. Laboratory data, not yet published, indicate that saturation places the aggregate in a vulnerable condition if it is to be exposed to freezing action. Air entrainment, of course, provides protection but the drier aggregates gave much better performance than those that were saturated. The proposed recommended practice applies regardless of the moisture content of the aggregate, but for good concrete control it is important that the moisture content of the aggregate be uniform.

The discussers state that they use as little fine lightweight aggregate and as much sand as possible and that fines are a problem in proportion to the amount used. The subcommittee recognizes that natural sand is rather commonly used to replace a portion of the lightweight fines to correct the grading and to improve workability and strength. However, high quality concrete can be obtained with many lightweight aggregates without the use of natural sand and with the advantage of lower unit weight. The specific

gravity and absorption of the fines are much more difficult to determine than for the coarse material, and variations in these determinations cause variations in the concrete mixes. The specific gravity factor obviates the need of these determinations.

The air dry unit weight less than 115 lb per cu ft refers to the unit weight specified in ASTM C 330 and this should have been stated more clearly in the subcommittee report. It is determined on a 6 x 12-in. cylinder moist cured 7 days and stored at 50 percent relative humidity 21 days.

As with all mix proportioning procedures, a detailed description becomes quite involved, but in actual use, the proposed recommended practice is almost identical to ACI 613-54 after the first mix has been made. Since determination of specific gravities is eliminated, the proposed recommended practice actually saves time and effort.

Discussion of a report by Subcommittee VIII, ACI Committee 325:

Design of Concrete Overlays for Pavements*

By J. D. LINDSAY and O. LARSEN, and SUBCOMMITTEE

By J. D. LINDSAY† and O. LARSEN‡

During the period 1930 to 1945, the Illinois Division of Highways constructed concrete overlays in numerous locations, providing in at least one case a direct comparison of the effects of variation in thickness of the overlay and of differences in treatment of the pavement before the overlay was placed.

While there is a good possibility that wartime restrictions, eliminating fabric reinforcement, dowel bars, and tie bars, may in some instances have influenced the condition of the overlays, particularly the one where direct comparisons were possible, observations over the years indicated the development of considerable roughness and distress due to climate and traffic, such as corner breaks and excessive transverse cracking, particularly in the thinner overlays, and it appeared that a thickness less than 8 in. did not generally prove satisfactory.

There was no easily discernible effect from a granular layer introduced between the pavement and overlay. Breakage of the pavement by means of a drop hammer before placing the overlay had no apparent effect, except that it did inhibit subgrade pumping in pavements susceptible to that condition.

Overlays appeared to have a substantially greater susceptibility to damage by weathering than pavments laid on the natural soil subgrade or on granular material. This is evidenced by disintegration, particularly along the center joint and transverse joints. It was present in the overlay on a 10-ft pavement but practically absent from the part that extended over the macadam shoulders. It is thought to be due to the frequent and prolonged presence of water in the joints and between the slabs, keeping the overlay saturated at these points and susceptible to damage by freezing. The use of air-entrained concrete, of course, had not been introduced at the time most of the overlays were placed.

Some studies have been made to observe if the quality of concrete in an overlay is affected adversely by the fact that it rests directly on a pavement slab rather than on earth or granular subgrade. The results, while they are not particularly conclusive, are presented because they may constitute significant information.

*ACI Journal, V. 30, No. 3, Sept. 1958 (*Proceedings* V. 55), p. 315. Disc. 55-19 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55).

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‡Engineer of Mixtures Control, Illinois Division of Highways, Springfield, Ill.

A number of cores were drilled from slabs representing a 10-in. overlay removed from a pavement in the course of construction operations, some from the part that had rested directly on the old pavement and some from the part that had rested on the adjacent granular subgrade. The dynamic modulus of elasticity was first determined on all the cores and some of them were tested immediately for their compressive strength. Others were subjected to 200 cycles of freezing and thawing with further determination of their dynamic moduli of elasticity, and then tested in compression.

The cores tested immediately, four from the part of the overlay that rested on the old pavement and four from the part that rested on the granular subgrade, showed the respective average dynamic moduli of elasticity of 4.9 and 5.7×10^6 psi, and the respective average compressive strengths of 4630 and 5823 psi. In view of this strength differential, it was decided to take 12 additional cores from the pavement remaining in place, six from above the old pavement and six from above the granular subgrade. The respective average compressive strengths were 4376 and 5633 psi. The lower value of the two may have been influenced by the fact that two of the cores represented had slightly reduced cross section at some point, caused in drilling. However, even if these are disregarded, the average strength of the remaining four would still be only 4722 psi. It seems, therefore, that there is a definite strength differential of the order of 20 percent, as based on the higher strength value, in favor of the part of the overlay placed on the granular subgrade.

The detrimental effect of 200 cycles of freezing in air and thawing in water was found to be greater for the cores taken from the overlay resting on the old pavement than for those taken from the part resting on the granular subgrade. The respective number of cores tested were three and four, and the respective average values of the dynamic moduli of elasticity before the start and after completion of the tests were 5.08 and 4.15×10^6 psi and 6.04 and 5.52×10^6 psi. The respective average compressive strengths were 3917 and 5616 psi. One core represented in each of these values again had slightly reduced cross section at some point, but it is felt that the two balance each other. The respective moduli are reduced to about 82 and 91 percent of the original moduli, and there is a strength differential of about 30 percent on the basis of the higher value.

The overlay, 6 years of age at the time of testing, was built with air-entrained concrete. Considering this and in view of the results obtained, it was decided to take cores from two additional overlays, one 14 years and the other 23 years of age, both representing non-air-entrained concrete. The 14-year overlay was 6 ft wider than the old pavement, the entire additional width extending to one side and resting on earth subgrade. The pavement was broken before placing the overlay. The 23-year-old overlay extended 5 ft beyond each edge of the old pavement, the subgrade being the shoulders of the old 10-ft pavement, probably predominantly earth.

Forty cores were drilled from each overlay, divided equally between the part directly above the old pavement and the part resting on soil subgrade. The respective strengths in case of the 14-year-old overlay were 5415 and 6035 psi, representing a strength differential of about 10 percent in favor of the overlay resting on the soil subgrade. In the case of the 23-year-old overlay, the respective values were 7519 and 7535 psi, indicating essentially no difference in strength. The results obtained, therefore, show similarity with those of the original tests only with respect to one of the older overlays. However, it is doubtful if any of the cores were taken in the close vicinity of distressed areas. It is felt that the data presented may warrant additional studies of the quality of the concrete in concrete overlays.

Certain consideration of the formulas for design of concrete overlays, set forth in the report, leads to the belief that they are based on the postulations that perfect contact exists between the pavement and overlay, and that the load and subgrade reaction force the two slabs into the same elastic curvature. Attention is called to the fact that these conditions may seldom, if ever, exist in actual practice.

The overlay unquestionably responds much faster than the underlying pavement to changes in temperature and other climatic conditions. Differentials in the degree of warping, therefore, are bound to exist and a continuous shifting of points of contact between the two slabs will take place, and perfect contact may well be the exception rather than the rule. Under such circumstances, a load on the overlay could not force the two slabs into the same elastic curvature.

Experimental evaluation of the formulas would nevertheless be of great interest. It also would be of interest to look into the possibility of using Westergaard's formulas for determining the stresses in concrete overlays, considering the underlying pavement as a subgrade with an exceedingly high modulus of subgrade reaction, even though these formulas are derived under the assumption of perfect contact between slab and subgrade.

Temperature warping stresses in concrete overlays must be rather high, considering that the underlying pavement, unlike an earth subgrade, is incapable of conforming to any degree with the curling movements of the overlay. The relatively concentrated contact areas between the two slabs, that must exist under this condition, also serve to increase the stresses due to load. A study of the combined load and temperature warping stresses that occur in concrete overlays, therefore, would be of considerable interest.

SUBCOMMITTEE CLOSURE

The discussion by Messrs. Lindsay and Larson contains valuable data, some of which confirm what has long been recommended in the construction of concrete overlays. It also points out the need for careful analysis of data and observations which cover as many of the factors involved as can be provided.

The conclusion that lack of durability in overlay concrete, as evidenced by disintegration along center joint and transverse joints, is due to presence of water in joints and between the slabs, seems of questionable validity. Observations of subgrades under concrete pavements have shown them to be continuously moist under all but the most extreme conditions, so that there is always a supply of moisture to the underside of the concrete—enough moisture to supply the concrete up to its full capacity of absorption. Without questioning the presence, at times, of moisture between the slabs and in joints, it does not seem possible that this could supply moisture to the concrete in excess of that to which any slab on natural subgrade has access.

Further, this hypothesis is not needed to explain the distress which seemed to develop more rapidly in the overlay than in concrete lying directly on natural soil or subbase materials. The tests reported by the discussers indicated clearly that the concrete in the overlay portion of that project had initial strength and dynamic modulus of elasticity substantially lower than the portion which rested on the granular subbase. The concretes going into the two parts were presumably of identical quality. What, then, was the difference?

It seems logical to conclude that the difference was the water-cement ratio at the time of hardening of the concrete. That concrete which overlaid the old slab had nearly the same water-cement ratio at the time of hardening that it had when placed. The concrete which lay on an absorbent granular subbase or on a natural soil lost enough of its initial mixing water by absorption by the subbase or natural soil to produce a lower water-cement ratio. The loss was enough to result in a substantially higher strength and modulus of elasticity in that concrete and also enough to provide a greater resistance to weathering. Or, stated conversely, the water retained in the overlay concrete caused an increased water-cement ratio at the time of hardening, sufficient to reduce materially the concrete's resistance to natural weathering processes.

This condition has long been known or at least suspected. It has been the basis for recommendation that concrete in overlay pavements be placed at the lowest possible water-cement ratio. The tests reported here confirm the validity of such recommendations.

It would be interesting to know the conditions of placement of the 23-year-old project on which there was no difference in strength between the overlay concrete and the widening concrete. Could it be that this concrete was placed as recommended—with such a low water-cement ratio that there was little or no difference in the loss of moisture? Or was the subgrade under the widening so well moistened that it would absorb no water from the concrete? On some resurfacing-widening projects, subgrade paper has been used on the widening portion to ensure uniform absorption conditions. The high strength, 7500 psi, of the concrete after 23 years would indicate that the first suggested explanation is probably the correct one.

As to excessive breakage noted as occurring on some of the projects: it is regretted that further data are not at hand. The assignment of Subcommittee VIII is to develop recommended practices for the design of concrete overlays. This assignment should be approached with an open mind and an effort made to arrive at a design procedure which will ensure an overlay adequate for the conditions under which it is placed and must serve, without being excessive in capacity and costing more than is necessary. If it develops that the subcommittee must recommend that overlays have a minimum thickness of 8 in. as suggested here, they should be prepared to make that recommendation. But before doing so, they should study the many projects which are of less than 8 in. in thickness and which have performed well, as well as those which Messrs. Lindsay and Larsen report to have given less than satisfactory performance. An effort must be made to determine the cause of the unsatisfactory performance, where it occurred, so that recommendations for design will include provisions to prevent recurrence.

The period 1930 to 1945, mentioned by Messrs. Lindsay and Larsen, is the period during which pumping was a major problem on concrete highways. Prior to 1930, traffic volumes and loads had seldom been heavy enough to cause pumping. By 1945, pumping had become the subject of several comprehensive studies which established the conditions under which pumping occurred and methods of prevention on future work.

During this period it is likely that many resurfacing projects were placed on old pavements which were pumping badly. As pointed out in the preliminary statement of Subcommittee VIII, a subgrade which has failed (and is pumping) will not be corrected by placement of an overlay slab. It is suggested here that a possible reason for the less than satisfactory performance noted by Messrs. Lindsay and Larsen may be found in pumping subgrades which were not corrected, as well as in the use of concrete mixtures not designed for use over a slab which will absorb little, if any, of the excess mixing water.

The discussion is valuable in that it tends to confirm the need for care in at least these two respects.

Discussion of a paper by B. Bresler and K. S. Pister:

Strength of Concrete Under Combined Stresses*

By C. J. BERNHARDT, MICHAEL CHI, A. COÜARD, and K. W. JOHANSEN

By C. J. BERNHARDT†

The test results presented by the authors are of great interest, and the new results can be considered as a valuable supplement to earlier research work in this field. They have worked out and presented their test results by considering the mean values of shear stresses and normal stresses after the method proposed by V. V. Novozhilov. This gives a possibility of determining the failure criterion in form of a certain curve in a τ-σ diagram. It will, however, also be of interest to work out the test results expressed by the principal stresses. Expressed in this form the test results can be compared directly with previous tests (especially with the tests reported by Wästlund[2]), also referred to by the authors. The test by Wästlund gives information on failure values in the section where both principal stresses are positive (compression), whereas the new tests concern values in the second quadrant of the principal stress diagram. The zone is limited by the line $\sigma_2 = -\sigma_1$. In Fig. A the new results are compared with those of Wästlund.

The principal stresses are then computed from the test results given in the paper, Table 6. The principal stresses are divided by the compressive strengths (3 x 6-in. cylinders, Table 5), to obtain dimensionless expressions. Further, to minimize the disagreement between measuring the compressive strength on relatively thin walled tubes and on 3 x 6-in. cylinders, all principal stresses are adjusted proportionally in the way that the mean values for $\sigma_1 = 0$ coincides at the point $\sigma_2/\sigma_a = 1.0$. σ_a then represents the compressive strength in this particular case. However, it must be assumed that it is impossible to obtain full agreement between the results from tests on specimens of different shape and size.

The results for the three different concrete qualities give no special trends and consequently can be presented by one mean curve valid for all three qualities. Expressed in this form the new test results offer a base for determining the limiting curve for the stresses in the material over a great area, which earlier has been subject to some doubt. Wästlund has indicated a straight line as the limiting curve in this area, and this straight line also has influence in the first quadrant, in that there must exist a uniform correspondence between the curves in the two quadrants. This straight line corresponds to

*ACI JOURNAL, V. 30, No. 3, Sept. 1958 (*Proceedings* V. 55), p. 321. Disc. 55-20 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55).

†Laboratory Engineer, Concrete Research Laboratory, Norwegian Institute of Technology, Trondheim, Norway.

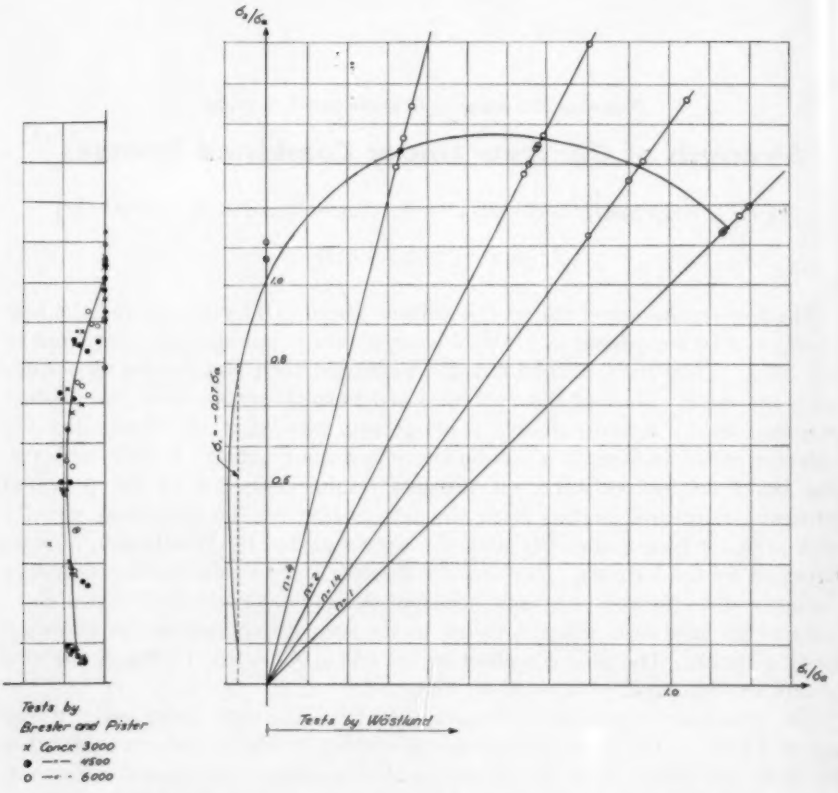


Fig. A—Results of Bresler and Pister compared with tests by Wästlund

failure after the hypothesis of maximum elongation. The limiting curve which can be drawn when the new results are compared with Wästlund's gives automatically a reasonably uniform correspondence at this point. This confirms the correlation between the test series, a fact that is worth emphasizing more, as the test specimens in the two series differ widely in shape and size. On the other side of the line, $\sigma_2 = -\sigma_1$, information is still lacking, except at the point where the limiting curve crosses the axis. At this point we will have to introduce the tensile strength of the concrete, but here also the problem concerning the shape and size of test specimens arises. It is possible that this specific problem of size and shape may explain the deviating form of the limiting curves presented by McHenry and Karni[3]) (see Fig. 3 and Fig. 8 in the quoted paper).

The form of the limiting curve (Fig. A) shows that the critical value of the principal tensile stress is relatively constant in the second quadrant. The influence of the compression is limited to a zone near the point $\sigma_2/\sigma_a = 1$.

For practical use of the failure criterion it would be of great importance if the limiting curve could be simplified to a straight line parallel the σ_2 axis. However, this simplification should not be allowed if a high degree of accuracy is needed. The curve in Fig. A offers the opportunity to establish the line $\sigma_1 = -0.07\,\sigma_a$ as a reasonable approximation. This line is dotted in the figure. It should be noted that this approximation can only be applied to values of σ_2/σ_a less than say 0.8.

APPLICATION OF FAILURE CRITERION

Development of diagonal cracks

The assumption that tension failure occurs in the concrete when the principal tensile stress has reached a certain critical value, allows a theoretical construction of the formation of diagonal cracks at different loading stages by bending of beams. This is of the greatest interest in connection with I-beams. A theoretical construction in this way for various series of prestressed beams has been carried out at The Concrete Research Laboratory, in connection with a research plan sponsored by the Norwegian Institute of Technical Research. This investigation* shows a good correspondence between the computed and the observed crack angles. The crack angles in the shear zone are found to group well around the value

$$\cot \psi = \sqrt{1 - \frac{\sigma_k}{\sigma_a}}$$

where ψ = angle between axis and diagonal crack; σ_k = normal stress in the concrete at the centroidal axis caused by the prestressing; and σ_a = tensile strength of concrete.

In this case:

$$\sigma_a = -0.07\,\sigma_a$$

$$\sigma_a = \text{compressive strength}$$

Fig. B shows the development of the diagonal cracks from a series of test beams at different load stages. In the same figure, the slope of the cracks corresponding to the computed values is also shown.

Calculation of ultimate load at shear failure

Messrs. Bresler and Pister give a method for computing the ultimate load at shear failure, and the method is employed on different reinforced rectangular beams without stirrups. An investigation of the shear failure of prestressed I-beams has been carried out at The Concrete Research Laboratory, Norwegian Institute of Technology, by applying failure criterion to the con-

*Bernhardt, C. J., "Diagonal Tension in Prestressed Concrete Beams," *Proceedings*, World Conference on Prestressed Concrete, San Francisco, Aug. 1957, pp. A 33-1–A33-6.

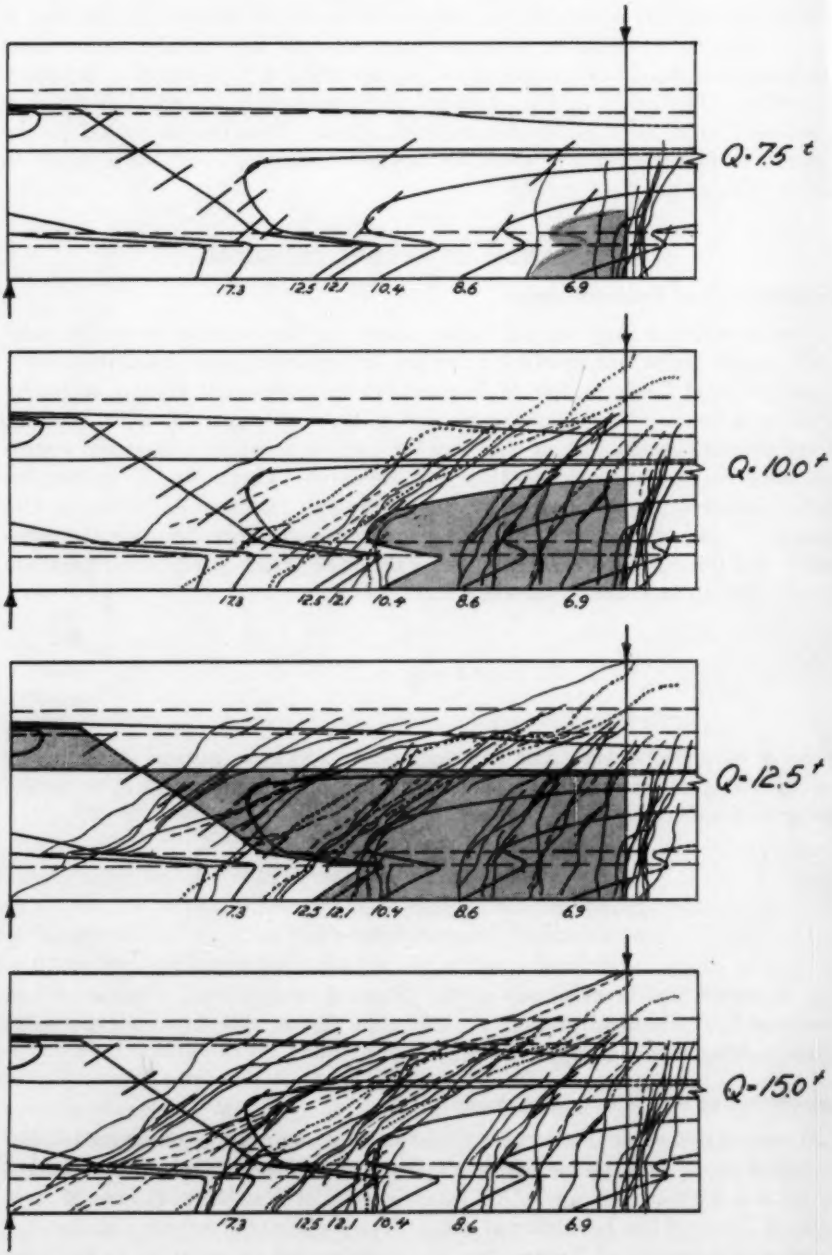


Fig. B—Development of diagonal cracks in a series of test beams at different load stages

crete in the flanges. The knowledge of the crack angles makes it possible to take into consideration the shear force carried by the stirrups. This inclusion in the theory of shear failure of the shear force carried by the stirrups, and the directions of the diagonal cracks, is necessary for obtaining a more complete knowledge of the shear problem. The base for this method is given by the three equations of equilibrium valid for the left part of the beam in Fig. C.

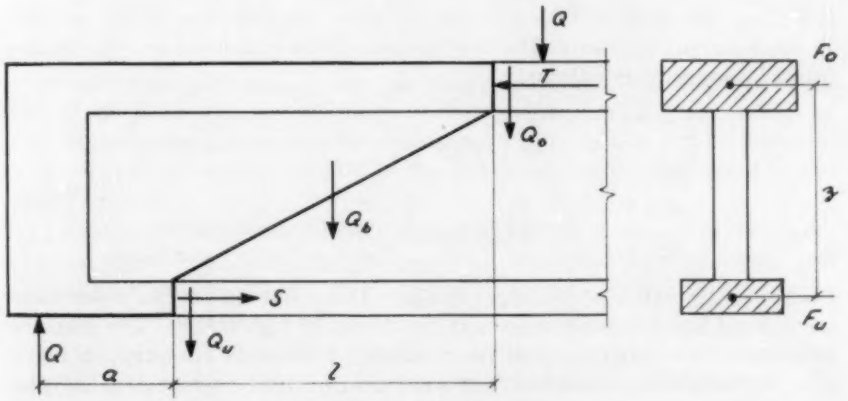
$$Q = Q_b + Q_o + Q_u$$

$$S = T$$

$$Tz = Q_b\,(a + 1/2\,l) + Q_u\,a + Q_o\,(a + l)$$

It is then assumed that the ultimate shear force is composed of the maximum force in the stirrups when the steel in the stirrups reach the yield point, together with the maximum shear force in the flanges. To determine the maximum shear force in the flanges it is necessary to make some simple assumptions for the distribution of the normal stresses and the shear stresses in each of the flanges. At this point it is assumed that there is uniform distribution of stresses, and the failure criterion is then applied to the mean shear stresses and mean normal stresses. This will give two new equations, one for each flange, so that, in all, five equations are at hand.

The problem, however, involves six unknown quantities. The missing equation can be found by studying the distribution of shear force between the



FK = Prestressing force
μ = Ratio of steel area to flange area
n = Ratio of moduli of elasticity
o = Index for upper flange
u = Index for lower flange

Fig. C—Prestressed I-beam investigated for shear failure

upper and the lower flange, and theoretically it should be possible to establish the equation by comparing the relative displacement of the two flanges, primarily considered on the left part of the beam, and secondly on the right part of the beam. However, to avoid this complication the simple assumption is made that the ultimate shear force in each flange is proportional to the areas of the flanges. This gives the equation in the simple form $Q_o/F_o = Q_u/F_u$. This assumption takes into consideration the main components of the relative displacement of the flanges. Thus it has been possible to reach a simple graphical solution of the equations by means of a general failure criterion given, for example, by Fig. A. An approximate hypothesis for the limiting curve may, however, also be used, without any serious influence on the accuracy of the solution of this complicated problem. The simplest way should then be to employ the maximum principal stress hypothesis. For the maximum principal tensile stress the value $\sigma_1 = -0.07\ \sigma_a$, may be used, and for the maximum compressive stress the value σ_a. Fig. D illustrates a graphical solution carried out in this way. We will then have two limiting curves in this diagram, one for failure in the upper flange and one for failure in the lower flange. Each of the limiting curves consists of two parabolas. This result is obtained by introducing the mean normal stress and shear stress, expressed by the resultant forces T, Q_o, and Q_u, into the equation

$$\frac{\sigma_1}{2} = \frac{\sigma}{2} \pm \sqrt{\left(\frac{\sigma^2}{2}\right) + \tau^2}$$

It is then necessary to take into consideration the influence of the prestress on the concrete stresses in the two flanges. Four equations for the limiting parabolas are then established:

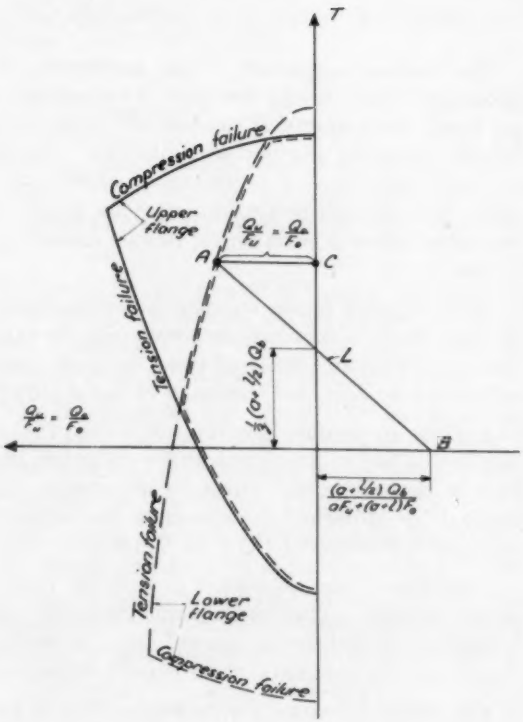
$$\left[\frac{Q_o}{F_o}\right]^2 = (0.07\ \sigma_a)^2 + \left[FK_o + \frac{T}{1 + n\mu_o}\right] 0.07\sigma_a \text{ Tension failure}$$

$$\left[\frac{Q_o}{F_o}\right]^2 = \sigma_a^2 - \left[FK_o + \frac{T}{1 + n\mu_o}\right] \sigma_a \text{ Compression failure}$$

These two are valid for the upper flange. The equations for the lower flange are derived by change of index and by change of sign for T. The graphical solution of the equations will be obtained by drawing the straight line L (Fig. D) which corresponds to the equation of internal moment. The position of the straight line will be determined by the amount of stirrups, the slope of the crack, and the shear span. The intersection between L and the limiting curve will give the solution in the way that the maximum possible value of $Q_o/F_o = Q_u/F_u$ is determined as the intercept AC.

There are four possible types of failure, corresponding to each of the two sets of parabolas, and the actual type of failure will be determined by the position of A. In Fig. D the type of failure should be tension failure in the lower flange. In this particular case, it is possible that the beam still will be

Fig. D—Graphical solution



able to carry greater loads through the upper flange and through the stirrups alone. An additional investigation is then required. This can easily be performed in a similar manner by putting $Q_u = 0$ and $F_u = 0$. In that case the limiting curve will be reduced to the two parabolas corresponding to the upper flange.

As mentioned above the analysis is carried out on the basis of maximum principal stress hypothesis; it will, however, be simple to replace the two parabolas with one single, uniform limiting curve for each flange, applying a failure criterion according to Fig. A.

The solution as in Fig. D has been applied to a great number of prestressed I-beams, with various degrees of prestressing and various flange areas. In most cases a close agreement between calculated and observed ultimate loads has been found. Mostly the position of Point A has been as shown in Fig. D. The additional investigation gives higher loads only where the value of F_o/F_u is larger than, say 2.

In all, the problem of ultimate load for shear failure is complicated, and it is believed that a method as described above will cover only a part of the possible combinations of shear span, amount of reinforcement, and the other quantities involved in the problem.

By MICHAEL CHI*

The authors' approach to the mechanism of failure of plain concrete is puzzling. While use of the three invariants of stresses, I_1, I_2, and I_3 may be justified, the property of unequal strengths in compression and in tension in brittle materials was not accounted for. Marin[13] found that this property must be taken care of by empirical coefficients and a shift of reference axes. Since Novozhilov's derivations were not given in full, further explanations are desirable before a criterion for ductile failure can be used directly for brittle failure.

Since concrete tensile strength is perhaps less than one-fifth of its resistance to pure shear, a torsional specimen may be expected to fail in principal tension. Such failures, referred to as "average shear" in Table 7, are misleading in spite of the fact that they may be equal numerically.

Applied to beams, Assumption 2 may be too conservative. Ability of reinforcing bar at joints to transfer shear has been a basis for design of dowel bars in pavements and crossbars in concrete hinges.[14] This ability was also implied by Mörsch.[15] Omission of this ability may contribute to the consistently low values of V_u/V_t in Table 9.

Criticism by the authors of $v = V/bjd$ based on "nominal effective area" is not justified. This formula was rigorously derived by basic mechanics for horizontal shear below a neutral axis. It would be justifiable if the authors assert that the horizontal shear is not a correct criterion for failure.

The above criticisms are perhaps minor in nature and should not mar the inspiring ideas and ingenious approaches presented in the authors' paper. Concept of resistance to shear in the uncracked portion of concrete beams is revolutionary. It may rationally explain failure of a test rigid frame[16] at $v = 72$ psi, and numerous failures of rigid frames in practice, which led to the recent revision of ACI Code for design against diagonal tension. One wonders whether the allowable nominal shearing stress of 100 psi in flat slabs is conservative enough at point of countraflexure. The authors' paper is the first step toward rational analysis of shearing resistance of beams.

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13. Marin, J., "Theories of Strength for Combined Stresses in Non-Isotropic Materials," *Journal of Aeronautic Sciences*, Apr. 1957, p. 265.

14. Reynolds, C. E., *Reinforced Concrete Designers Handbook*, Concrete Publications, Ltd., London, 4th edition, 1948, pp. 124 and 129.

15. Hognestad, E., "What Do We Know About Diagonal Tension and Web Reinforcement in Concrete?" *Circular* No. 64, University of Illinois Engineering Experiment Station, Fig. 1, p. 13.

*Member American Concrete Institute, Associate Professor of Civil Engineering, The Catholic University of America, Washington, D. C.

16. Wilson, W. M., *et al.*, "An Investigation of Rigid Frame Bridges," *Bulletin* 308, University of Illinois Engineering Experiment Station, 1938, Part II.

By A. COÜARD*

We must express our gratitude to the authors for having renewed these experiments on torsion-compression, and thus to confirm their previous experimental results which had been so valuable to us in checking our new theory of strength of materials.

We have been impressed by the fact that the authors seem not to have found any experimental result showing a reduction of the torsion strength (Fig. 7) when axial compression increases. This result is in perfect accord with the Viest experiments on shear, showing an arch for the shear crack.

As the discusser wrote† "The shear cracks . . . show that the concrete area needed to resist shear diminishes as the compressive stress increases, which is in perfect contradiction with the results which could be predicted from the application of Mohr's circle and the intrinsic curve."

The elastic theory is developed logically from certain simple assumptions. If the end results of the elastic theory can be proved false, then either the basic hypotheses or the reasoning, or both, must be incorrect. We believe that we have proved the elastic theory wrong on both counts, and since 1953 we have been awaiting experimental or theoretical challenge (or confirmation) of our views.

By K. W. JOHANSEN‡

In continuation of my comments to McHenry-Karni paper on the same theme[17] I should like to make a short remark. When the new tests are plotted with σ_m/σ_c as abscissa and τ_m/σ_c as ordinate we get Fig. E. Here σ_c is the strength in pure compression. From triaxial compression tests[18,19] we know that the corresponding plotting gives a slightly curved line which near the point corresponding to the pure compression test may be approximated by a straight line of inclination 0.6. The continuation of this line is shown in Fig. E. Corresponding to the cleavage rupture we get another straight line with inclination 1.0 as the condition for this failure is

$$\sigma_3 = -\sigma_t = \tau_m - \sigma_m$$

where σ_t is the tensile strength. This line is drawn through the mean-value point on the τ_m-axis in Fig. E. The tests agree fairly well with these straight lines. When the tests by McHenry and Karni are treated in the same manner we again get fairly good agreement with the same straight lines.

*Member American Concrete Institute, Civil Engineer, Paris, France.
†Coüard, A., *Nouvelle Conception de la Résistance des Matériaux* Génie Civil, Paris, 2nd edition, 1957, p. 21.
‡Professor, Structural Research Laboratory, Technical University of Copenhagen, Copenhagen, Denmark.

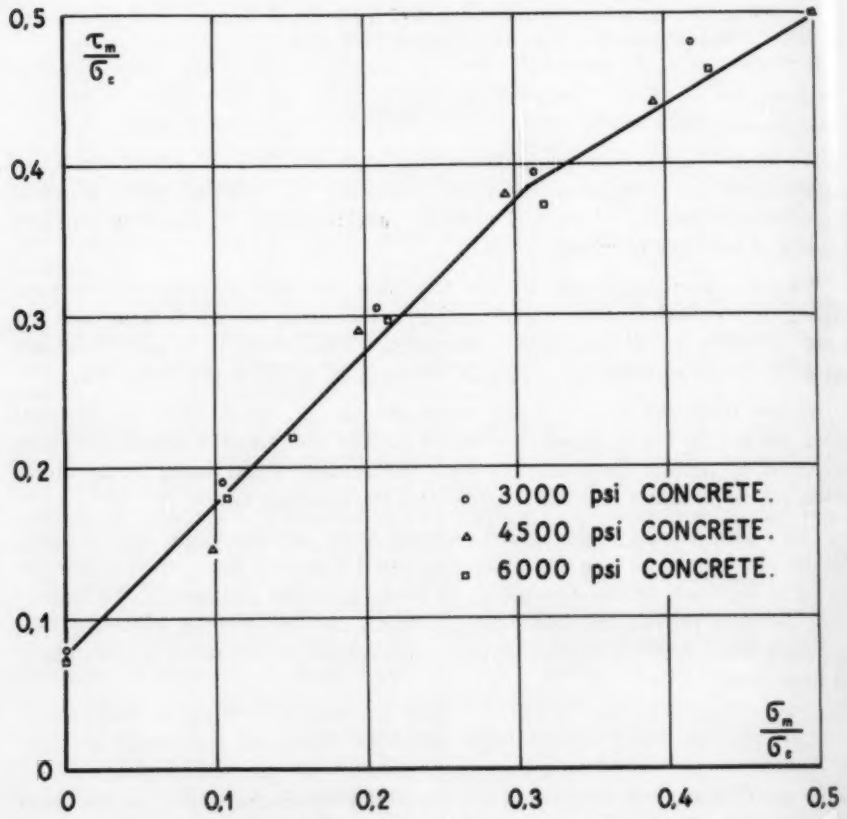

Fig. E—Authors' results plotted in terms of σ_m/σ_c versus τ_m/σ_c, where σ_c is strength in pure compression

The equation of the first line is

$$\frac{\tau_m}{\sigma_c} - 0.5 = 0.6\left[\frac{\sigma_m}{\sigma_c} - 0.5\right]$$

or

$$\sigma_1 = \sigma_c + 4\sigma_3$$

Richart, Brandtzaeg, and Brown[20] got the same equation with 4.1.

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17. Johansen, K. W., discussion of "Strength of Concrete under Combined Tensile and Compressive Stress," ACI JOURNAL, V. 30, No. 6, Part 2, Dec. 1958 (*Proceedings* V. 54), pp. 1303-1305.

18. *Report* SP-23, Structural Research Laboratory, Denver, Colo., 1949.
19. Richart, Brandtzaeg, and Brown, *Bulletin* 185, University of Illinois Engineering Experiment Station, 1928.
20. Richart, Brandtzaeg, and Brown, *Bulletin* 190, University of Illinois Engineering Experiment Station, 1929.

AUTHORS' CLOSURE

The authors wish to express appreciation to all the discussers for their comments. Mr. Chi stated that the property of unequal strengths in compression and tension in brittle materials was not accounted for. Apparently the table following Eq. (13) on p. 337 was overlooked. On the basis of a linear relation between mean compressive and shearing stresses the relative values of uniaxial shear and tensile strength are given in terms of f_c'.

Mr. Chi's defense of $v_{max} = V/bjd$ as a rigorously derived expression cannot be accepted. Considering the freebody shown in Fig. F and assuming the so-called rigorous shear stress distribution, the shear V_s carried by the longitudinal reinforcement across the crack and the shear V_c carried by the compression zone can be obtained as follows:

$$V_s = v\,b\,(d - kd) = \frac{V_u}{bjd}\,bd\,(1 - k) = V_u\,\frac{1 - k}{j}$$

and

$$V_c = V_u - V_s$$

Using approximate values of k and j of $\frac{3}{8}$ and $\frac{7}{8}$ respectively, one finds $V_s = 5V_u/7$ and $V_c = 2V_u/7$. It is indeed difficult to conceive that 5/7 of the total ultimate shear is transferred by dowel action. Preliminary tests,* while not conclusive, indicate that at loads approaching ultimate the magnitude of V_s is usually less than 15 percent. Thus the values obtained from "rigorous" shear stress formulas lead to completely erroneous results. This anomaly arises from the fact that the "rigorous" derivation of $v = V/bjd$ is based on fallacious assumptions of stress distribution in a cracked reinforced concrete beam.

The nominal shearing stress $v = V/bjd$ can be used as a design criterion, but it may lead to erroneous results if it is considered to be a rigorously derived quantity.

Mr. Bernhardt's comparison of the present test results with those obtained by Wästlund[2] is indeed gratifying. As pointed out the differences in shape and size of test specimens required to produce biaxial stress states in different stress quadrants are complicating factors in attempting to correlate various experimental data.

Mr. Bernhardt's application of failure criteria to determination of shearing strength of prestressed concrete I-beams is novel and interesting. As he

*Watstein, D., and Mathey, R. G., "Strains in Beams Having Diagonal Cracks," National Bureau of Standards, Washington, D. C., 1958.

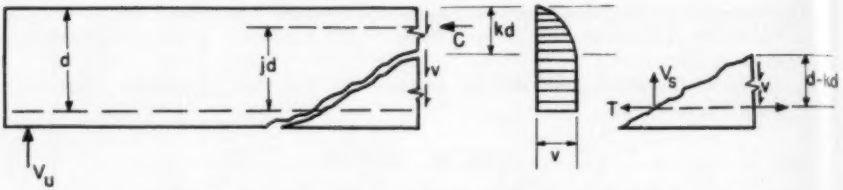

Fig. F—Shear pattern and free-body diagram at cracked section

points out, the assumptions made in establishing the failure envelopes and the empirical definition of crack direction limit the validity of the method to a particular class of beams.

Dr. Johansen, following the theory of Mohr, suggests the use of maximum shearing stress $\tau_m = (\sigma_1 - \sigma_3)/2$ and normal stress $\sigma_m = (\sigma_1 + \sigma_3)/2$ in presenting combined stress test results in preference to mean shearing stress and mean normal stress as used in the present paper. The difference is centered in the significance of the intermediate principal stress σ_2. Novozhilov[4] has shown that the ratio of mean shearing stress to maximum shearing stress satisfies the inequality

$$0.731 \geqq \frac{\tau_a}{\tau_m} \geqq 0.633$$

Thus, over the range of all possible states of stress the two definitions of shearing stress lead to quantitative differences not larger than 15 percent. The major effect of the intermediate stress is found in the normal stress. For biaxial states of stress, as pointed out previously by Johansen, no information can be gained since, with $\sigma_2 = 0$,

$$\frac{\sigma_a}{\sigma_m} = \tfrac{2}{3} = \text{constant}$$

and the difference in the two stresses is only a detail of definition. It is only in triaxial states of stress that the possible influence of σ_2 can be ascertained. Unfortunately, very few tests are available to substantiate one or the other approach.

In concluding the authors would like to emphasize that the choice of mean stresses is predicated upon certain concepts that appear to be significant: (a) sufficient generality to include biaxial as well as triaxial states of stress, (b) provision for the possible influence of the intermediate principal stress, and (c) independence of the properties of elasticity, isotropy, or homogeneity of the material.

Discussion of a paper by Edward K. Rice:

Economic Factors in Prestressed Lift-Slab Construction*

By RICHARD C. CLARK, JAMES R. LIBBY, IRWIN J. SPEYER, and AUTHOR

By RICHARD C. CLARK†

Mr. Rice has made a valuable analysis of prestressed lift slab costs in concrete structures. His conclusions compare quite favorably with the recent field experience of Western Concrete Structures Co., Inc., subcontractors and equipment manufacturers in prestressing and lifting.

Since his paper was initially presented at the ACI 10th regional meeting in 1957, subcontract prices of prestressed lift slabs have continued to decrease at the same rate as the slopes of the respective curves shown in Fig. 2. This has been possible in our own experience during 1958 largely because of increased efficiency in fabrication of the tendons on medium and large jobs.

Further decreases in lifting prices are anticipated beginning 1959-60 due primarily to significant design improvements in new lifting gear now being field tested. These changes are expected to lower labor and lifting costs on some of the types of lifting gear used and licensed within the industry.

As Mr. Rice notes, economies in the structure are the result of simplicity. For example, one can make all columns and collars to the same sections, regardless of small to moderate differences in loads. In particular, more widespread use can be made of the shear key connection, illustrated in Fig. 10, as opposed to welding. Collars of 100 percent fabricated steel can save up to half the cost of castings.

There are distinct bidding advantages in using standardized shear keys, collars, columns, forms, and appurtenant hardware. The mass production and open-stock inventories long used on tendon hardware, anchorages, etc., are now being used on these miscellaneous iron "accessories."

Finally, the interim financing must not be overlooked quantitatively in the case of private work. Three months saved on a 100,000 sq ft building costing $1,600,000 amounts to thousands of dollars of interest saved, and if the building betters the owner's investment by a net 6 percent, 3 months of income can be equated to 24 cents per sq ft.

*ACI Journal, V. 30, No. 3, Sept. 1958 (*Proceedings* V. 55), pp. 347-358. Disc. 55-21 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55).

†Member American Concrete Institute, Western Concrete Structures Co., Inc., Gardena, Calif.

By JAMES R. LIBBY*

The author has presented an interesting discussion of the factors affecting the economy, feasibility, and design philosophy of prestressed concrete lift slabs. His contribution to the art of lift-slab construction has been significant and worthy of note, since it is apparent that his efforts in collaboration with others have resulted in lifting techniques and devices which have materially reduced the cost of this construction.

The cost information which was presented in Table 1 would lead the casual reader to believe that prestressed concrete and reinforced concrete lift-slab structures were being constructed in California as early as 1952. The writer represented a prominent prestressing firm in the western United States and Canada from 1952 through 1955 and cannot recall any prestressing work in connection with lift slabs in the western states prior to late 1953, with the exception of experimental construction in Texas in 1952 and 1953 by the Southwest Research Institute. Late in 1953 a school in Henderson, Nev., which used prestressed lift slabs, was designed and put out for bids by a Las Vegas architect.† One of the earliest prestressed concrete lift-slab structures in California was the *Evening Outlook* Newspaper Building in Santa Monica, constructed in 1954, which had inverted beams on the roof slabs from which the second floor was suspended in order to obtain a large column-free area on the ground floor.

Due to his past activities as a lift slab contractor the author is eminently qualified to compile a chronological list of early prestressed lift-slab structures with their respective bidding dates. Such a list would certainly be of value to the profession now and in the future years when it may be advantageous to compare these structures with other structures which have utilized different construction methods in order to determine the relative merits of each type of construction. The writer earnestly hopes the author will undertake this service to the profession in his closing remarks.

The experience and records of the writer do not reflect the reduction in the cost of prestressing materials from $0.85 per lb in 1952 to $0.55 per lb in 1957 as is indicated in Table 1, and which accounts for 90 percent of the total indicated reduction in the cost of this construction during this period. Although the unit cost of post-tensioning materials is significantly affected by the quantity, size, and length of the individual tendons, even for the very early and small jobs which were bid in California and Oregon in 1952 and 1953 the cost of the prestressing materials rarely exceeded $0.70 per lb for the successful post-tensioning method. On one large job on which the writer was intimately associated in mid-1953 in the Los Angeles area, the cost of prestressing materials installed, stressed, and grouted was about $0.60 per lb in spite of the fact that the tendons had an average length of approximately 44 ft and about half of the cables were small.

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†Peterson, J. L., "History and Development of Precast Concrete in the United States," ACI JOURNAL, V. 25, No. 6, Feb. 1954 (*Proceedings* V. 50), pp. 491-494.

A significant lift-slab job* was constructed in Winsted, Conn., in 1956 which utilized conical type anchors rather than button heads. This is pointed out as a means of clarifying the author's remark which could be misconstrued and conceivably lead the reader to conclude that the headed wire systems are the only ones which are practicable for this type of construction. The normal size conical type anchors have been successfully applied to slabs as small as 6 in. thick while the very small size could be used on thicknesses as little as 4 in. The one-wire systems which have only recently been introduced in this country as well as the bar systems could also be applied to this type of construction successfully and economically under many conditions.

GROUTED TENDONS

The author should amplify his statement pertaining to the "inconsistent and often poor results" which have been obtained when grouted tendons are used in lift-slab construction in such a manner that the specific difficulties which have been encountered are accurately described. This statement is ambiguous and is certainly contrary to recommended prestressing practice in nearly all European countries and is contrary to the results of experiments which have been conducted by numerous researchers throughout the world as can be found in even a cursory study of the literature pertaining to prestressed concrete. A thorough study of this literature will certainly convince the opened minded engineer of the advantages to be gained as a result of proper grouting.

There is no need for the writer to enumerate the many real advantages which are known to result from grouting; however, it should be pointed out that the lack of flexural bond in nongrouted construction seriously increases the width of cracks which are obtained if the cracking load is inadvertently exceeded, and the lack of bond materially reduces the ultimate moment that a section could develop if it were adequately grouted. These are extremely significant considerations in partially prestressed construction in which tensile stresses are allowed in the concrete under full load and where ultimate moment computations are relied upon to a more significant degree in determining the adequacy of the safety factors. The use of partial prestressing is described by the author as often good practice in prestressed lift slabs.

In discussing "Tentative Recommendations for Prestressed Concrete," Janssonius† states ". . . As I understand it, there is no grouting of prestressing cables or bars in an unbonded construction. I believe that unbonded prestressed elements should be avoided and should not be mentioned in the

*Minges, James S., and Wild, Donald S., "Six Stories of Prestressed Slabs Erected by the Lift-Slab Method," ACI JOURNAL, V. 28, No. 8, Feb. 1957 (*Proceedings* V. 53), p. 751.

†Janssonius, G. F., discussion of a report by ACI-ASCE Committee 323: "Tentative Recommendations for Prestressed Concrete," ACI JOURNAL, V. 30, No. 3, Part 2, Sept. 1958 (*Proceedings* V. 54), p. 1231.

recommendations." Although the writer feels this viewpoint is too strong and that unbonded tendons should be allowed in less important structures and where fatigue and flexural bond stresses are relatively insignificant, it is important to recognize that there are professional engineers who feel the use of unbonded tendons yield questionable results.

The opinions expressed by T. Y. Lin,* to the effect that designers should think in terms of the adequate performance of structures in the so-called elastic range and be certain of the factors of safety of the structures, are shared by the writer since computations for an inelastic material such as concrete obviously cannot be precise if based upon elastic assumptions. Because of this, stress distributions at design and cracking loads as determined by elastic analyses are at best approximate—a fact which substantiates the need of the very real advantages which are to be gained from grouting with a properly proportioned, mixed, and injected grout.

An adequate recommended practice for grouting has not as yet been written in this country. Although the remarks contained in "Tentative Recommendations for Prestressed Concrete" pertaining to grouting constitute a good starting point, the writer believes from his own experience that these recommendations should be expanded to include limits for the water-cement ratio to be used in the grout mixture, recommendations regarding the use of aggregates and admixtures, and limits for the pressure used to force the grout through the ducts. It should also be strongly recommended that the ducts which contain the post-tensioned tendons be water-tight from entrance port to exit port before grouting is commenced as a means of preventing the leakage of grout and the plugging of ducts and sheaths which result from such leaks. This last point is practically never covered by grouting specifications and can mean a great deal in the success of the grouting operation. Experience has shown that with an adequately written grouting specification, competent inspection, and adequate equipment and workmanship, the "inconsistent and often poor results" can be changed to a very real improvement in an unbonded structure at negligible increase in cost.

By IRWIN J. SPEYER†

The author's review of various factors relating to prestressed lift slabs, which was most interesting and informative, has considerable authority by virtue of his experience record as a former contractor combined with his present consulting partnership. While the paper only considers practice and economies in the Southwest, the conclusions drawn should apply to other areas as well. There are certain items which it would indeed be interesting to have the author enlarge upon, some of which have been dealt with in the paper, and others which have not.

*Lin, T. Y., discussion of a report by ACI-ASCE Committee 323: "Tentative Recommendations for Prestressed Concrete," ACI JOURNAL, V. 30, No. 3, Part 2, Sept. 1958 (*Proceedings* V. 54), p. 1232.

†Member American Concrete Institute, Freyssinet Co., New York, N. Y.

DESIGN OF FLAT SLABS

Under service loading

The two general methods employed for determining service load moments are:

1. By coefficients, as in ACI 318-56
2. By a bent analysis
   a. Using a uniform width strip in each direction.
   b. Using a varying width strip, such as is formed by diagnonals emanating from the columns.

The three methods have been compared in a previous ACI paper and differences worthy of consideration have been noted. The writer's past practice has been Method 2(b), primarily because this avoids duplicating portions of the same load in two directions. Perhaps the author would care to comment on his own practice.

Under ultimate loading

The author advocates the use of nonbonded tendons. The recent ACI-ASCE Joint Committee 323 recommendation states in effect that unless tests have been performed on structures closely approximating the proposed construction, the steel stress at ultimate should be considered as only 15,000 psi higher than the stress at service loads. For the steel which the author uses, this represents roughly a 10 percent increase over design stress as compared to perhaps a 50 percent increase when using bonded tendons. We wonder whether the author has knowledge of any tests to ultimate failure of nonbonded prestressed flat slabs, and what his practice is with respect to computing the ultimate strength of this type of construction.

CONSTRUCTION PRACTICES

Precast concrete columns

The author's use of precast columns is a logical step in the development of lift slab construction, and his opinion that long columns are more economical when prestressed is shared by others. The major problem when employing concrete columns, that of connection to slab, has been solved in one fashion by the author (Fig. 10). With respect to this detail, it is not clear whether the bottom of the concrete slab rests on top of the steel shear key, or whether a connection is made within the space between the column and the slab, resulting in the bottom of the slab being flush with the bottom of the key.

This detail raises a second question. The problem of resting a precast slab which is continuous over a number of supports on these supports, with the assurance that all points of bearing are at the same elevation, is one to which the author has probably given considerable thought. Possibilities of fabrication errors in the columns exist and permissible tolerances in erection must be allowed for. For instance, the possibilities of an out-of-plumb

column and a column slot which has not been manufactured truly perpendicular to the column might result in the slab bearing on one side of a shear key only. The author's assurances would be indeed welcome.

Prestressing

The author indicates a preference for nonbonded tendons and states that grouting has yielded "inconsistent and often poor results." In the past 8 years during which time prestressing has exhibited its amazing rate of growth and acceptance, we believe it a conservative estimate to state that over 100,000 bonded (grouted) post-tensioning cables have been employed in the United States alone. Cases of inconsistent and even poor grouting have been only a negligible percentage of the total, and can be directly attributed to procedures not in accord with standard recommendations, or to careless supervision. This is not to say that we can accept today's standard procedures as being the final answer, and the time spent by the various professional and manufacturing organizations on this question is proof indeed of their continuing interest. It is our feeling that this widespread interest and large number of research projects is directly attributable to the opinions of the majority that a properly bonded tendon is preferable to a nonbonded one. In the minds of many engineers, important questions relating to behavior of nonbonded prestressed members have not fully been answered, such as the safety of this type of construction when subjected to fire. Results of a fire test would go a long way toward answering this question. If test data are lacking, perhaps the author would indicate his opinion on this problem.

In closing, we believe we also speak for other engineers in stating that the author's firm should be publicly congratulated for the efforts which they have expended over these past years toward the proper development of this new form of construction. Theirs has been a major contribution to the present state of the art of prestressed lift slab construction.

AUTHOR'S CLOSURE

The author is pleased by the interest shown in the paper and the discussions which have followed its publication. The discussions have raised several points which deserve comment at this time.

Design

Since the design of prestressed concrete lift slabs is the subject of a forthcoming paper by the author and Felix Kulka,* detailed discussion of the design of prestressed concrete lift slabs will not be undertaken at this time. However, in answer to Mr. Speyer's query regarding the design of flat slabs, we prefer using the bent analysis because it is relatively simple and yields satisfactory results. The results of recent tests conducted on a continuous prestressed lift slab at the University of California, sponsored by the California

*Presented at ACI 55th annual convention, Los Angeles, Feb. 25, 1959.

Division of Architecture, have shown this method of analysis, when properly applied, to be satisfactory for service loads.*

For convenience of construction as well as analysis, the rectangular pattern of tendons is preferred to the diagonal pattern.

Nongrouted tendons

It is well known that the grouting of tendons will slightly increase the ultimate strength of a prestressed concrete member but in prestressed concrete lift slabs, the design is controlled by proper behavior—which is generally a flat slab with negligible deflections. Such a slab when properly designed will possess more than sufficient ultimate strength even with nongrouted tendons, as has been shown by several tests at the University of California. Tests have shown that due to the number of ups and downs in a continuous tendon the high friction developed produces a slab with an ultimate strength almost as great as could be obtained with a well grouted tendon.

Hence, under all the conditions encountered so far, there has been no necessity of grouting for increasing the ultimate strength in these slabs.

Fire resistance of nongrouted tendons

A recent fire test on a two-way slab prestressed with unbonded tendons clearly demonstrated the tremendous fire resistance of this type of construction. The test was conducted by Prof. George Troxell of the University of California in accordance with standard ASTM test procedures. The unbonded tendons had $1\frac{1}{2}$ in. coverage of hardrock concrete. The test specimen technically "failed" in 3 hr, 50 minutes due to heat transmission to the unexposed surface; however, it continued to carry the load for a full 4 hr!

Construction practices

Due to the low depth-span ratio of most lift slabs, they are quite flexible and some unevenness of support is not serious. Originally it was thought that shimming to take up variations between the shear key and the lifting collar might be necessary; however, to date little if any shimming has been required.

*Lin, T. Y., Scordelis, A. C., and Itaya, R., "Behavior of a Continuous Concrete Slab Prestressed in Two Directions," Institute of Engineering Research, University of California, Aug. 1958. Also presented at ACI 55th annual convention, Los Angeles, Feb. 26, 1959.

Discussion of a paper by Harold Samelson and Abba Tor:

Stresses in Reinforced Concrete Sections Subject to Transient Temperature Gradients*

By TUNG AU†

The authors have presented a practical procedure for determining the thermal stresses in reinforced concrete sections subject to transient temperature gradients. There are several points in the paper which the writer would like to clarify.

In Fig. 3, the steady state gradient indicates a change of temperature from 220 F at interior face to 211 F at exterior face of concrete. Actually, there is a temperature drop of $1.87 \times 2.36 = 4.4$ deg through the 1 in. insulation, according to the computations shown on p. 379. The change of temperature in concrete is therefore from 215.6 F at interior face to 206.8 F at exterior face. However, this does not affect the subsequent computations for steady state gradient since Δt for concrete remains 8.8 deg.

To make $C = T$ in Fig. 4, a line perpendicular to the wall surfaces must be so located that $A_1 = A_2$. A general formula can be given for a concrete section with n equal segments. With reference to Fig. A, let $y_0, y_1, y_2, \ldots y_n$ be the transient temperatures at 0, 1, 2, . . . n segments from the interior face of concrete. Then, from the areas of the trapezoids, it can be seen that the temperature y_a at the neutral axis from such temperature gradient is

$$y_a = \frac{1}{n}\left[\frac{y_0}{2} + y_1 + y_2 + \ldots\ldots\ldots + y_{n-1} + \frac{y_n}{2}\right] \ldots\ldots\ldots (13)$$

If Δt in Eq. (4) is defined as the difference between y_a and the temperature y at any fiber of the concrete section, or $\Delta t = y_a - y$, the circumferential stress at this fiber will be given by:

$$f = E \alpha \Delta t = E \alpha (y_a - y) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Eq. (14) will indicate automatically tensile or compressive stress since f is positive for $y < y_a$ and is negative for $y > y_a$. It should also be noted that A in Eq. (5) and (6) has the unit of deg-in. and is not the same A as defined

*ACI JOURNAL, V. 30, No. 3, Sept. 1958 (*Proceedings* V. 55), p. 377. Disc. 55-23 is part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55).

†Member American Concrete Institute, Associate Professor of Civil Engineering, Carnegie Institute of Technology, Pittsburgh, Pa.

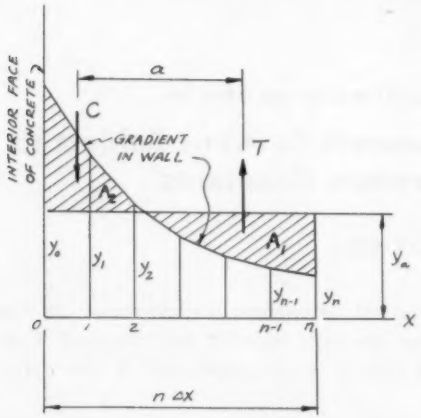

Fig. A—Temperature gradient in terms of transient temperatures at 0, 1, 2, ...n

in the notation on p. 377. With α having the unit of 1/deg, the unit of C or T is force per unit length of the tank in its axial direction as given by Eq. (5):

$$C = T = \text{deg-in.} \times \text{lb per sq in.} \times 1/\text{deg} = \text{lb per in.}$$

And with the distance a in in., the unit of moment M_t as given by Eq. (6) is:

$$M_t = \text{lb per in.} \times \text{in.} = \text{in.-lb per in., or ft-lb per ft}$$

Applying Eq. (13) to the example shown in Fig. 5, $y_0 = 168$ F, $y_1 = 121.7$ F and $y_2 = 83.7$ F for $n = 2$. Hence,

$$y_a = \tfrac{1}{2}\,(168/2 + 121.7 + 83.7/2) = 123.8 \text{ F.}$$

The difference of $123.8 - 121.7 = 2.1$ F is slightly different from the value of 1.3 F indicated in the figure. Furthermore, it is clear from the geometry of the areas shown in Fig. 5 that the area A from either tensile or compressive force is smaller than $45 \times 12.5 \times \tfrac{1}{2}$, and the distance a is not exactly $2/3 \times 25$ as given in the computation for critical transient temperature gradient on p. 382. While these numerical errors are rather insignificant insofar as affecting the final result is concerned, they may obscure the fundamental relations in Eq. (6).

More often than not, stresses instead of moments from the external loads are superimposed on those resulting from the temperature gradient. Then the circumferential stress at any fiber due to the temperature gradient can readily be computed by Eq. (14). For instance, at interior face of concrete, $f = E \alpha (y_a - y_o)$, or

$$f = 3 \times 10^6 \times 5.5 \times 10^{-6} \times (123.8 - 168)$$

$$= -729 \text{ psi compression in concrete.}$$

Discussion of a paper by J. A. Hanson:

Shear Strength of Lightweight Reinforced Concrete Beams*

By JOHN E. BOWER, PHIL M. FERGUSON and J. NEILS THOMPSON, I. M. VIEST, and AUTHOR

By JOHN E. BOWER†

The writer would like to supplement the author's comments and test results with results of his own thesis investigation.‡

The diagonal tension cracking data of the author's beams were analyzed using an expression developed by the writer. In the analysis only the initial diagonal tension crack is considered. The equation

$$\frac{v_c}{\sqrt{f_c'}} = 1.425 + \frac{6 + 39\,b/d}{\dfrac{(M/V)_c}{npd} + 10}$$

which is a modification of that presented by Morrow and Viest,[6] was developed from a series of tests comprising five simple beams and 29 simply supported beams having various degrees of moment restraint at the ends. All beams were of Wabash River gravel and torpedo sand.

Fig. A shows that there is good agreement between the author's results and the aforementioned equation. The figure is especially effective in pointing out the increased diagonal cracking strength observed by the author for those beams using Aggregate 7. It is these results and the absence of a definite trend in the remaining results which preclude the addition of a "lightweight aggregate reduction factor" to the writer's equation. It should be added that the writer's equation agrees well with the data of Morrow and Viest[6] and Moody, *et al.*[4]

Two companion specimens in the author's tests gave conflicting results, one failing by diagonal tension cracking while its companion failed at a much higher load than that producing diagonal cracking. This phenomenon was also noticed by the writer in four pairs of companion specimens designed to fail at a higher load than the diagonal cracking load. In all cases, it was noted that the diagonal tension crack in the beam failing at the low diagonal

*ACI JOURNAL, V. 30, No. 3, Sept. 1958 (*Proceedings* V. 55), p. 387. Disc. 55-24 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE; V. 30, No. 9, Mar. 1959 (*Proceedings* V. 55).

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‡Bower, John E., Jr., "Shear Strength of Restrained Concrete Beams Without Web Reinforcement," MS Thesis, Department of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill., Aug. 1957.

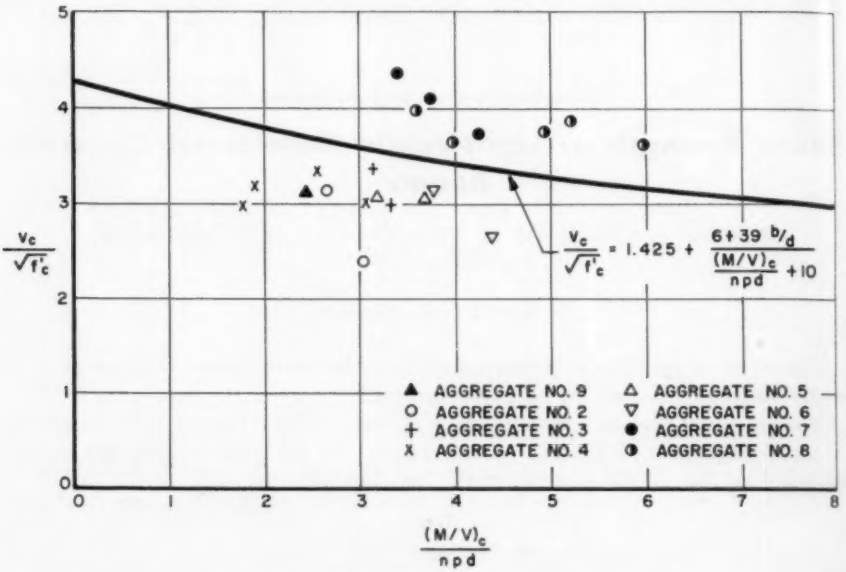

Fig. A—Comparison of author's diagonal tension cracking data with Bower equation

cracking load was less steep than the crack in the companion specimen which failed at a load greater than that producing diagonal cracking.

It is concluded that the aforementioned equation adequately predicts the diagonal tension cracking load of lightweight concrete beams. Also, from the observation that, given two companion beams, one may fail at the formation of the diagonal tension crack while the other sustains additional load, it is evident that the diagonal tension cracking load should be considered the ultimate load in any design calculations so long as the beam is without web reinforcement.

By PHIL M. FERGUSON and J. NEILS THOMPSON*

Although investigations on lightweight beams should certainly be encouraged, it is unfortunate that Conclusions 3 and 4 were included in this paper. It is the opinion of the discussers that Conclusion 3 is refuted by a more careful analysis of the data presented and Conclusion 4 is possible only because the author limited himself to small (impractical) shear spans and large percentages of steel.

The conclusions as stated in the paper seem to indicate that the ACI Building Code values of allowable shear stress are amply safe when used with lightweight concrete. Evidence is accumulating that such is not the case.

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Since A. P. Clark[7] in 1951 called attention to the increased shear resistance which went with loads close to the support, the ratio of shear span to depth, usually designated as a/d, has been considered a major variable in diagonal tension studies. When a/d exceeds 4 or 5 the ultimate shear v is nearly constant and the exact value of a/d is not too important. When a/d is as small as the author's value of 2.5, strength of ordinary gravel concrete is roughly 60 percent higher than for an a/d of 4. When a/d is only 1.35, the shear strength is tripled or quadrupled.[8]

This idea of a general and large increase in shear strength as a/d decreases has had wide acceptance for a number of years in spite of the fact that the exact a/d beyond which changes in shear strength become quite minor is not yet accurately mapped for all conditions. The value is probably a function of the percentage of tension steel; and it may not be the same for gravel concrete and for lightweight concrete.

Many of the shear tests made earlier (and even some after 1951) must be largely ignored or discounted because they used small a/d values. To these discussers it appears that the data presented by the author have small practical value for the same reason. Lower ultimate strengths would have been found if larger a/d values had been used in the program; and it is these lower values which must be used in establishing permissible design stresses for shear.

There is, however, an important feature introduced in this paper which is significant. The author uses the diagonal *cracking* load as the basis for shear strength. This is a step in the right direction. It means that the shear strengths which the author reported will not be influenced as severely by a change in a/d as would be the case if he had used ultimate shear strengths. Nevertheless, there is no reason to assume that diagonal tension cracking is independent of a/d. There is considerable evidence to show that initial diagonal cracking is also a function of a/d.

The author compares his results with a series of sand-and-gravel beams reported by Moody, *et al.* In the fourth paper[9] in that series (p. 701) a formula for v_c at initial diagonal cracking is developed. This formula includes the factor $(1 - 0.1M/Vd)$, where M/Vd is equivalent to a/d. This would indicate that an a/d of 4 would lead to a unit shear only 80 percent as large as those for the author's sand-and-gravel beams with a/d of 2.5. The difference for lightweight beams might be larger.

A second variable, not so universally recognized but nevertheless quite important, is that of the percentage of tension steel. The author's beams each contained 2.5 percent of steel. Tests at the University of Texas indicate that shear strength (for $f_c' = 3500$ psi) decreases about 15 percent for each 1 percent decrease in the steel ratio.

The author's test results have been plotted in Fig. B, which is similar to Fig. 7 in the original paper except that the beams with 5.0 percent steel have been omitted, the *second* diagonal cracking stress has been omitted,

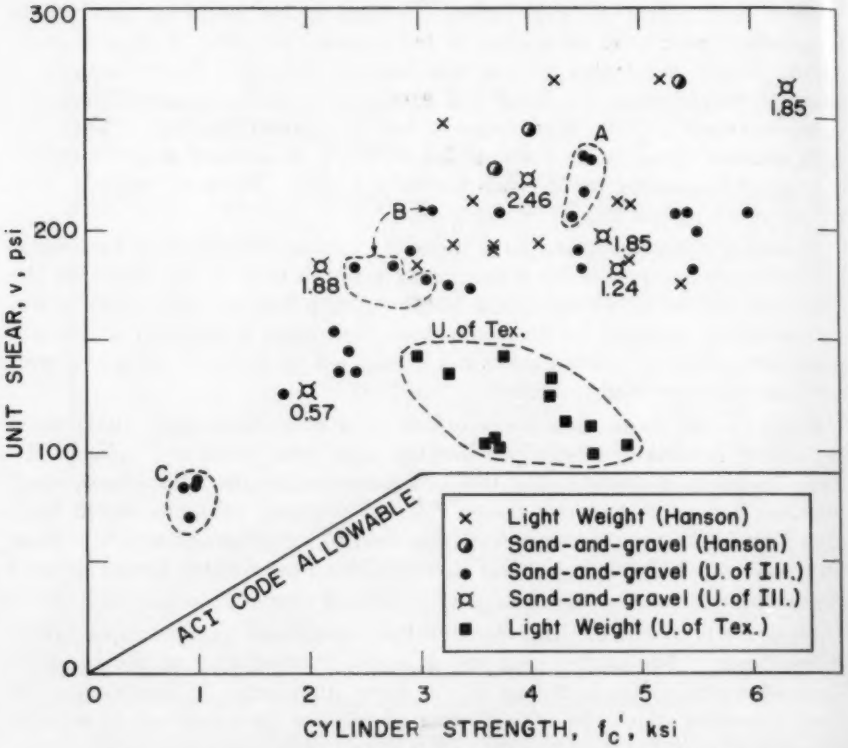

Fig. B—Revised plot of shear strength versus cylinder strength

and a cross mark has been used for all the various lightweight aggregates. For purpose of discussion, three groups of the University of Illinois sand-and-gravel beams have been indicated as Groups A, B, and C, while all the other University of Illinois beams that are denoted as black circles may be considered as constituting a fourth group, D. A fifth group, E, also from the University of Illinois, includes five beams denoted by open circles with external cross marks. The author compared all these five groups with the lightweight beams, although not separately.

In making such a comparison, it must be kept in mind that all the lightweight beams contained 2.5 percent steel and were tested with an a/d of $26/10.5 = 2.47$. For the sand-and-gravel beams the corresponding data are different, as indicated in Table A. (The last line of data refers to some University of Texas tests on lightweight beams which are mentioned later in the discussion.)

Since none of these University of Illinois beams contained as much steel as the author's beams, it is not surprising that generally their shear strengths

are low compared to the author's results. In contrast the single University of Illinois beam with 2.46 percent of steel (at f_c' about 4 kips per sq in.) falls quite near the author's sand-and-gravel beams (with 2.5 percent of steel).

TABLE A—COMPARISON OF p AND a/d FOR BEAMS OF FIG. B

Group	p, percent	a/d
Hanson	2.50	2.47
A (University of Illinois)	2.15—2.37	2.92 to 3.06
B (University of Illinois)	1.60—1.66	
C (University of Illinois)	0.80—0.82	
D (University of Illinois)	1.89	3.41
E (University of Illinois)	0.57—2.46 as noted on Fig. B	1.89 to 2.02
University of Texas	0.75—1.50	3.50 to 7.00

Furthermore, the beams in Groups A, B, C, and D had a/d values considerably larger than the author's beams. This should, and did, lead to lower shear strengths. In Group E the smaller a/d would be expected to give higher strengths but this was offset by the small steel percentages (marked on Fig. B) used in all but one of this group.

The above study points out that the comparison which the author proposes between his lightweight beams and the University of Illinois sand-and-gravel beams is rather far fetched and actually should carry little weight. These discussers feel that Conclusion 3 should simply have quoted from an earlier comment in the paper:

> "Results for concrete made with lightweight aggregates No. 2-6 and No. 9 are grouped closely at approximately 75 percent of the shear strength of the normal weight concrete made with Aggregate 8. Aggregate 7, the sintered shale, produced concrete with a shear strength equal to that of Aggregate 8."

For practical use, the validity of any comparisons established at an a/d of 2.47 is somewhat doubtful.

When a/d is increased and the steel is reduced to the usual design range, all types of concrete show up less favorably in shear strength. Out of data accumulated from graduate thesis research at the University of Texas, but not yet ready for publication, 12 lightweight concrete (expanded shale aggregate) beams have been found which were tested at a/d values of 3.5 to 7.0 and contained percentages of steel ranging from 1.5 percent down to 0.75 percent. These beams all failed in diagonal tension, with f_s below the yield point. Their *ultimate* shear strengths have been represented by solid squares in Fig. B. (Ultimate shear strength is not much, if any, above the cracking strength in the case of the larger a/d values.) These plotted points demonstrate rather specifically that Conclusion 4 in the paper, which mentions safety factors of 2.0 to 4.0, does not apply widely enough to be used as a general guide.

It is not difficult to envision practical cases, even without the presence of any external restraint, in which the factor of safety would be as low as 1.15 or 1.20. It thus appears that stirrups probably should be required in all lightweight concrete beams or that the unit shear stress should be kept much below the 90 psi limit in the ACI Building Code.

REFERENCES

NOTE: References are numbered consecutively with the author's original references.

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9. Moody, K. G., and Viest, I. M., "Shear Strength of Reinforced Concrete Beams. Part 4—Analytical Studies," ACI JOURNAL, V. 26, No. 7, Mar. 1955 (*Proceedings* V. 51), pp. 697-732.

By I. M. VIEST*

Numerous experimental investigations of reinforced concrete beams during the past decade have shown that the diagonal tension strength of a section depends primarily on four quantities: (1) the cross-sectional dimensions, (2) the quality of concrete, (3) the percentage of tensile reinforcement, and, (4) the ratio of moment to shear. General conclusions concerning the comparative behavior of beams made with lightweight aggregates may be drawn only after the coverage of all four of these important variables. Thus, other investigations are needed in addition to that reported by the author. Meanwhile his data should prove valuable in checking the available theories for diagonal tension cracking and in planning of future experiments.

The author chose the concrete quality as the only major variable; he also included two levels of the percentage of tensile reinforcement but kept constant the ratio of shear to moment and the cross-sectional dimensions. By inclusion of five specimens made with sand-and-gravel concrete, the author made it possible to evaluate *directly* the comparative behavior of lightweight aggregate beams. The data show clearly that the diagonal tension strength varies with the type of aggregate and that the diagonal tension strength of lightweight beams may be significantly lower than that of sand-and-gravel beams. On the other hand, one series shows that lightweight beams may be equally strong or even stronger in diagonal tension than sand-and-gravel beams.

The variability among the beams made from different type of aggregates is not surprising particularly in view of Shideler's report[3] which has shown rather wide variations in the structural characteristics of the concretes included in the author's study. This variability points out the need for an analytical approach which would make it possible to evaluate diagonal tension strength on the basis of certain known properties of any particular type of concrete. Until such an approach is developed, it will be necessary to determine the diagonal tension strength of every lightweight aggregate concrete separately through an extensive experimental study.

Theoretical considerations suggest that concrete may affect the diagonal tension strength through four of its characteristics: the resistance to prin-

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cipal tensile stress, the modulus of elasticity, the bond characteristics, and the development of tensile cracks. Experiments with sand-and-gravel concretes have shown that for such aggregates a reasonably good correlation of the test data may be obtained by expressing the concrete quality with one value, cylinder strength f_c'. This finding suggests that for sand-and-gravel concretes the variations in the effects of bond and tensile cracking are negligible.

For concretes of lightweight aggregates the relationship between the tensile strength and the modulus of elasticity may be significantly different than for sand-and-gravel concretes. Thus, data such as those reported by the author permit a particularly valuable check on the existing theories for diagonal cracking.

The writer's recent studies have indicated that diagonal tension strength of a cross section may be evaluated approximately from the following expression:

$$\frac{V}{bd} = \left(A + B\frac{E_s}{E_c}\frac{Vpd}{M}\right) f_t' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

V = external shear force at the section considered

b, d = dimensions of the cross section

E_s, E_c = moduli of elasticity of steel and concrete, respectively

V/M = ratio of shear to moment at the section considered

p = ratio of tension reinforcement

f_t' = resistance of concrete to principal tensile stress

A, B = numerical parameters

For beams made with sand-and-gravel concrete, Eq. (1) gives safe values of the ultimate strength in diagonal tension with

$$Af_t' = 1.9\sqrt{f_c'} \text{ psi}$$

$$B\frac{E_s}{E_c}f_t' = 2500 \text{ psi}$$

The diagonal tension strength of author's beams made with Aggregate 8 was computed from Eq. (1) and a comparison with the test results is shown in Table B.

Thus on the average, the author's beams of sand-and-gravel concrete were 20 percent stronger than indicated by Eq. (1).

The strength of beams made with lightweight aggregates cannot be computed from Eq. (1) because the values of A, B, and f_t' are not known. How-

TABLE B—DIAGONAL TENSION STRENGTH, TEST RESULTS COMPARED WITH CALCULATED VALUES, FOR BEAMS MADE WITH AGGREGATE 8

Specimen	V_{test}/V_{calc}
8A-X	1.21
8A	1.26
8B	1.24
8C	1.16
8D	1.11
Average	1.20

ever, if the assumption is made that Parameters A and B are the same as for sand-and-gravel concrete, the ratio of the tensile resistance f_t' to the tensile resistance f'_{tsg} of the sand-and-gravel concrete may be evaluated from Eq. (1) with the aid of the test data as follows:

$$\frac{f_t'}{f'_{tsg}} = \frac{V_{test}}{1.2\,bd\sqrt{f_c'}\left[1.9 + 2500\left(\frac{E_{csg}}{E_c}\right)\frac{Vpd}{M\sqrt{f_c'}}\right]} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where $V_{test} = 0.5\,P_{S1}$.

E_{csg} = modulus of elasticity of sand-and-gravel concrete

E_c = modulus of elasticity of lightweight concrete

$M/V = a - d = 15.5$ in.

Values of P_{S1} are given in Table 4. The modulus of elasticity E_{csg} is given in Table 3 for five values of f_c'; the following relationship is in excellent agreement with the data:

$$E_{csg} = 48{,}500\sqrt{f_c'} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

For every type of aggregate, E_{csg}/E_c, computed with E_{csg} from Eq. (3) and E_c values from Table 3, was found to be almost constant; the average ratios were used for each type of aggregate.

Ratios f_t'/f'_{tsg} computed from Eq. (2) are shown in Table C.

TABLE C—f_t'/f'_{tsg} COMPUTED BY EQ. (2)

Specimen	f_t'/f'_{tsg}	Specimen	f_t'/f'_{tsg}
9C	0.64	5A	0.82
		5B	0.74
2A	0.64		
2B	0.68	6A	0.77
		6B	0.68
3A	0.75		
3B	0.73	7A-X	1.01
		7A	0.97
4A	0.67	7B	0.94
4B	0.67		
4C	0.51		
4D	0.51		

The actual ratios f_t'/f'_{tsg} may be estimated reasonably accurately on the basis of the moduli of rupture. Unfortunately, such data are not reported by the author and Shideler's study[3] contains moduli of rupture only for specimens subjected to continuous moist curing. However, the ratios in Table C are of reasonable order of magnitude except perhaps for Specimens 4C and 4D.*

If the ratios in Table C are correct values for the materials under consideration, the differences in diagonal tension strength of the lightweight and sand-and-gravel beams are fully explained by the differences in the tensile strength and in the modulus of elasticity. Eq. (1) with

$$A = \frac{1.9\sqrt{f_c'}}{f'_{tsg}}$$

$$B = \frac{4.0\sqrt{f_c'}}{f'_{tsg}}$$

*Placing an arbitrary limit $V_{calc}/bd \leq 3.0\sqrt{f_c'}$ results in $f'_t/f'_{tsg} = 0.64$ and 0.69 instead of 0.51.

could then be used with reasonable confidence for predicting the diagonal tension strength of beams made with lightweight aggregates.

On the other hand, if the ratios in Table C are not correct, either the Parameters A and B are functions of the quality of concrete (bond characteristics, tensile cracking), or Eq. (1) is not generally applicable.

AUTHOR'S CLOSURE

The author sincerely appreciates Mr. Bower's discussion that confirms the parity of lightweight concrete shear resistance and that of normal weight concrete. The discussion is particularly useful because the analysis includes the effect of widely varying moment-shear ratios and tension steel percentages. In accordance with Morrow and Viest,[6] Mr. Bower has expressed the relationship between diagonal tensile strength and compressive strength as the ratio of unit shear at cracking load to the square root of the compressive strength. The effects of moment and amount of tension steel on shearing resistance are expressed by the term

$$\frac{(M/V)_c}{npd}$$

$(M/V)_c$ is the ratio of moment around the centroid of the compressive force to the shear at the section of diagonal tension cracking, i.e., it is equal to a_c, where a_c is defined as the distance from the section of zero moment to the intersection of a diagonal tension crack with the tension reinforcement. For most test beams reported in the literature, a_c would be assumed as half the shear span. More will be said of this type of analysis later in this closure.

It is highly gratifying to read both Mr. Bower's and Professors Ferguson's and Thompson's agreement that only the diagonal cracking load should be considered as the ultimate in design of non-web-reinforced beams.

Professors Ferguson and Thompson question the accuracy of the author's Conclusions 3 and 4. Their objections are based on two features of the test program: (1) beams with relatively short "shear span," and (2) relatively high tension steel percentages. The author was certainly aware at the outset of the program that these variables, particularly the first, affect nominal unit shear to a moderate but significant degree. However, it was the purpose of the investigation to gather shear strength data on concretes using seven representative lightweight aggregates, rather than to investigate the whole field of diagonal tension. The deliberate omission of the terms "shear span" and "a/d" from the paper might have caused some misunderstanding of this limitation of the program. Such terms are unfortunate in the study of diagonal tension since they have significance only in the case of the most simple concentrated loadings.

Evidence of the validity of their objections is offered by Professors Ferguson and Thompson by their Fig. B. In this plot of nominal unit shear versus compressive strength, they have reproduced the data of the author's

lower strength beams and a few chosen points from the data of References 4 and 6. To this, they have added the unpublished results of 12 lightweight concrete beams tested at the University of Texas. The shear strengths of all 12 of these beams, containing a *single* lightweight aggregate, fall below those reported in the paper.

In spite of the "more careful analysis" presented by the discussers, it is not clear to the author just how this "study points out that the comparison which the author proposes between his lightweight beams and the University of Illinois sand-and-gravel beams is rather far fetched and actually should carry little weight." Neither is it clear why only these particular data from the above mentioned references should be considered as proper comparison with the lightweight concrete data. Instead, Fig. B exhibits the large scatter of results common to all diagrams of nominal shear versus compressive strength. The addition of the University of Texas data strongly enhances the author's remarks early in the paper regarding the large variation in structural characteristics of lightweight aggregates, even between aggregates of similar type.

Professors Ferguson and Thompson recommend rejecting or discounting diagonal tension data from beams with a/d values less 3.5 to 4.0. The author prefers to consider that tests within this range disclose important inherent characteristics of the material, just as do comprehensive tests of 6 x 12-in. cylinders. The extension of such data to the wide range of a/d values used in practice is another problem—and an important one—on which many investigators are now concentrating attention.

The discussers surmise that the reduction of diagonal tension strength with increasing moment-shear ratio may be larger for lightweight concretes than for normal weight materials. To clarify this point, the author has prepared Fig. C, using the parameters proposed by Mr. Bower. The use of these parameters by the author should not be construed as indicating complete agreement with their particular formulation. However, the trends of the three important variables, compressive strength, M/V, and tension steel percentage, are certainly in the proper direction. Thus, the parameters are adequate for present purposes. The University of Texas results, which were made available to the author by Professor Ferguson, have been added to the author's to give a range of data for lightweight concrete beams with a/d values from 2.48 to 7.0 and steel percentages from 0.75 to 5.0. The more voluminous data on normal weight concretes from Reference 6 and from Mr. Bower's thesis have been superimposed on the same diagram. The Texas beams and the PCA lightweight beams (with the exception of the higher-strength No. 7) overlap Bower's beams, and the three sets of data from a continuous sequence which suggests strongly that there is no significant difference in the manner in which the strengths of the various materials vary with M/V or with steel percentage.

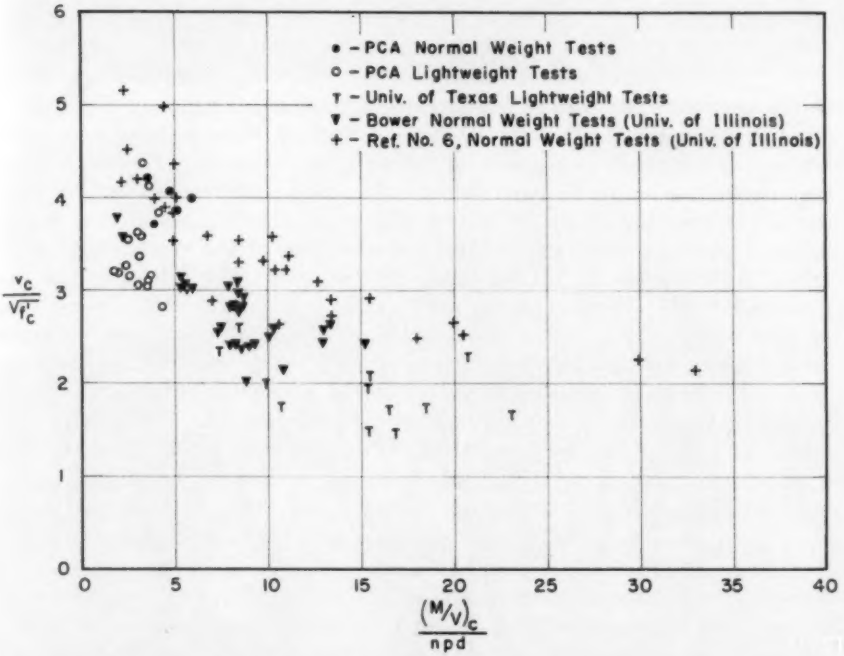

Fig. C—Bower parameters for shear strengths of lightweight and normal concrete beams

Fig. C also shows that the author was warranted in comparing his data with those of other investigators. The Texas beams certainly fall within the lower range of the data, but much of Bower's data are within the same range. Thus, Conclusion 3 of the paper is justified and should be included without revision. Some discussion may be in order in regard to the last sentence of Conclusion 3 and to part of the text of the paper. The above discussion and Fig. C involve the unit shearing resistance due to applied dead and live loading. If extraneous tensile stress, such as that from restrained drying shrinkage, is superimposed on that due to applied load, the indicated resistance to diagonal tension will be lowered. Some, but not all, lightweight concretes exhibit greater drying shrinkage than most normal weight concretes. In cases where restraint will occur with these higher shrinkage concretes, conservative allowable shear values should be employed in design.

Concerning Conclusion 4, it certainly was not the purpose of the investigation to study the adequacy of the current ACI Building Code in regard to shear stresses. The conclusion only stated the obvious results of Fig. 7 *in regard to the beams studied at the Portland Cement Association Laboratories.* In light of the comparison shown by Fig. C, it appears that lightweight con-

crete is in the same relative position as normal weight concrete insofar as factors of safety over the Building Code are involved.

The author appreciates the discussion by Dr. Viest. There is one point in his discussion that causes considerable concern on the part of the author. That is the suggestion that the modulus of rupture might be used as a measure of the resistance of concrete to principle tensile stress. The modulus of rupture test, in which the nonreinforced specimen is allowed to dry for some period, is sensitive to the effects of shrinkage. During the drying period residual tensile stresses may build up at the faces of the specimen, or even actual visible cracking may develop. Both conditions would reduce the indicated tensile strength by variable and uncertain amounts. In the case of a reinforced concrete beam that undergoes the same drying period, much of this nonpredictable strength reduction is probably removed by the presence of the reinforcing steel. The steel should reduce the differential stresses set up by the moisture gradient within the concrete and at the same time cut off any cracks that tend to develop in the tension fibers of the beam.

The author has computed the shear resistance for his beams by the formula for lightweight concrete suggested by Dr. Viest. The ratio, f_t'/f'_{tsg}, was obtained from modulus of rupture test specimens that were cured and stored under the same conditions as the beams. The comparisons of measured diagonal tension resistance to the calculated value, V_{test}/V_{calc}, ranged from 1.30 to 2.30 with an average value of 1.71. These large deficiencies in the calculated shear strengths indicate rather conclusively that the ratio of modulus of rupture of lightweight concrete to that of normal weight concrete is not acceptable for the purpose of predicting ultimate shear resistance.

ACI News Letter

Vol. 30, No. 9 JOURNAL of the AMERICAN CONCRETE INSTITUTE March 1959

Detailing Practices

Buildings to Bridges

58 Illustrative Drawings

The ***Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-57)*** incorporates under one cover the previous separate editions on detailing of buildings and highway structures. It is a correlation of the latest improved methods and standards for preparing drawings for the fabrication and placing of reinforcing steel. Sections on detailing and fabricating shop practice are translated into practical examples in typical drawings. Spiral bound to lie flat, it is the only publication of its kind in English and is meeting wide acclaim among designers, draftsmen, and engineering schools.

Full Price $4.00 **ACI Members** $2.50

AMERICAN CONCRETE INSTITUTE

PUBLICATIONS

P.O. Box 4754, Redford Station Detroit 19, Mich.

ACI

News Letter Contents

March 1959

On the Cover—Apartment building in Tannenbusch bei Bonn, Germany, features an exposed reinforced concrete frame. Built in 1951, the structure is 12 stories high with two floors below ground. Over-all dimensions in plan are 20 x 20 m. Architects were Apel, Letocha, and Herdt. Hochtief A. G.-Zublin A. G. served as engineer and contractor. Owner of the building is the High Commission of Germany, Frankfort.—*Photo courtesy Deutscher Beton-Verein E.V., Wiesbaden*

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Annual Report:

1958—A break-even year

Both ACI income and expenses increased during 1958 to record totals with the final results for the year being a net of $693.42—a virtual balance between income and outgo. Strength in publications sales, which increased 20 percent over 1958, offset a late-year weakness in receipts from membership dues. However, membership continued its slow but steady growth with a net addition of 500 members to bring the year-end total to 9723.

Strong contributors to publications sales were JOURNAL subscriptions, separate prints, the *Manual of Standard Practice for Detailing Reinforced Concrete Structures*, and the *ACI Building Code*. Principal new publication was the *Concrete Primer*, an up-dated version of Past President McMillan's best-seller which first appeared in 1928.

Highlights of the year were occupancy of the new headquarters building and its dedication in October. Many members had an opportunity to tour the building at that time and visitors from all parts of the world have been numerous. It has truly served to call attention to concrete as an imaginative building material.

AMERICAN CONCRETE INSTITUTE
ASSETS, LIABILITIES, AND RESERVES

December 31, 1958

ASSETS

General Fund		
Cash on hand		
Michigan Bank	$28,635.55	
Detroit Bank and Trust Co.	12,500.00	
Community Federal Savings and Loan Association	10,000.00	
National Bank of Detroit, Investment Account	424.07	
		$ 51,559.62
Accounts receivable		
Advertising	2,688.17	
Publications	15,093.71	
		17,781.88
Investments, U. S. Government Bonds		80,000.00
Prepaid expenses		745.43
Furniture and Fixtures	33,064.25	
Less reserve for depreciation	10,311.78	
		22,752.47
Land and building, at cost		315,687.05
TOTAL ASSETS, General Fund		488,526.45

ASSETS continued

Retirement Fund		
Cash	$ 470.15	
Investments		
U. S. Savings Bonds	49,500.00	
Stock, Standard Oil of N. J.	4,387.50	
TOTAL ASSETS, Retirement Fund		$ 54,357.65
Henry C. Turner Trust Fund		
Cash	408.99	
Stock, American Telephone and Telegraph	2,500.00	
TOTAL ASSETS, Turner Trust Fund		2,908.99
Alfred E. Lindau Trust Fund		
Cash	508.06	
U. S. Savings Bonds	4,000.00	
TOTAL ASSETS, Lindau Trust Fund		4,508.06
COMBINED TOTAL ASSETS		$550,301.15

LIABILITIES AND RESERVES

Accounts Payable		$ 14,888.43
Reserve for Detailing Manual		4,636.13
Mortgage Payable		55,255.21
Retained earnings to Dec. 31, 1958	172,155.69	
Cost of building and furniture transferred from Building Endowment Fund	241,590.99	
		413,746.68
TOTAL LIABILITIES, General Fund		488,526.45
Retirement Fund Reserve		54,357.65
Henry C. Turner Trust Fund Reserve		2,908.99
Alfred E. Lindau Trust Fund Reserve		4,508.06
COMBINED TOTAL LIABILITIES		$550,301.15

COMPLETE COVERAGE OF THE 55th ANNUAL ACI CONVENTION WILL APPEAR IN THE APRIL ISSUE

RECEIPTS AND EXPENDITURES—1958

RECEIPTS	
Membership dues	$134,629.58
Membership certificates	407.35
Publication sales	128,049.10
Advertising sales	33,440.07
Interest	2,529.27
Miscellaneous	1,606.84
Sales tax	7.81
TOTAL INCOME	$300,670.02

EXPENDITURES	
Personnel	$ 98,282.61
Member service (including JOURNAL)	80,856.86
Publications (other than JOURNAL)	61,249.55
Advertising expense	16,537.60
Office operation	26,747.83
External activities	7,321.29
Building operation	11,213.91
	$302,209.65
Less discount earned	−2,233.05
NET EXPENDITURES	$299,976.60
Excess of income over expenses	$ 693.42

ASTM to organize new division on materials sciences

A new Division on Materials Sciences will be organized by the American Society for Testing Materials to coordinate and intensify the development of knowledge of the fundamentals of materials. The division will concern itself with the collection, establishment, and publication of basic information essential in creating a better understanding of materials and their properties.

Development of the division has been entrusted to K. B. Woods, Purdue University, president; F. L. LaQue, The International Nickel Co., Inc., and A. Allan Bates, Portland Cement Association, vice-presidents; and Robert J. Painter, executive secretary.

General inspector school conducted at Anchorage

Over 100 construction inspectors attended the inspector schools and seminars conducted the latter part of January at the Anchorage headquarters of the U. S. Army Engineer District, Alaska.

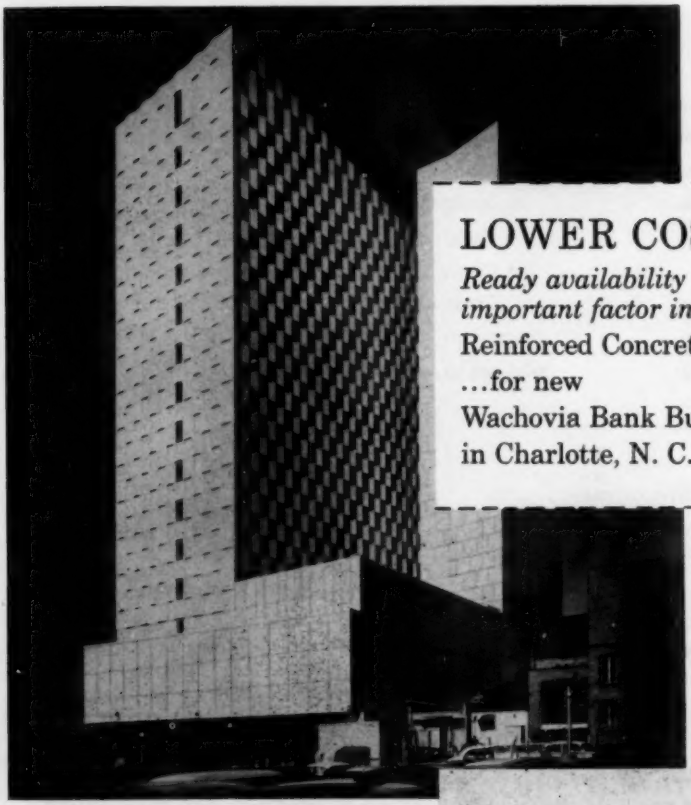
Corps of Engineer inspectors and representatives from Alaska Communications System, USARAL, Alaska Air Command, and Federal Aeronautics Authority personnel from all parts of the state attended the sessions devoted to mechanical, electrical, welding, asphalt paving, concrete, and soils inspectors.

Plan 3rd International Congress of the Concrete Stone Industry

The Directors of the International Bureau of the Concrete Stone Industry recently announced that the 3rd International Congress of their group will be held in Stockholm in 1960.

The German speaking member Associations of the International Bureau (Austria, Switzerland, and Germany) have been allotted the theme "The Shrinkage of Concrete" (lightweight concrete, heavy concrete, cast stone, and prestressed concrete).

A general report of this subject will be compiled under Wolfgang Czernin Dipl. Engineer, who is anxious to secure pertinent reports from experts on this subject so that he can incorporate them into the general report. Material should be forwarded Mr. Czernin at Forschungsinstitut des Vereins österreichischer Zementfabrikanten, Vienna III, Reisnerstrasse 53, Austria, to reach him by June 1, 1959.



LOWER COST...

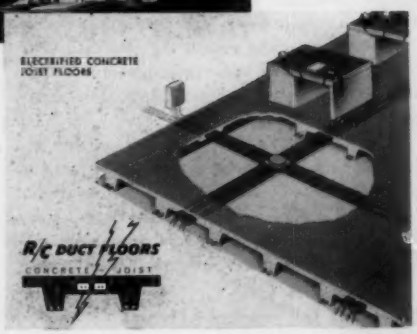
Ready availability of materials important factor in choice of
Reinforced Concrete framing
...for new
Wachovia Bank Building
in Charlotte, N. C.

Wachovia Bank and Trust Company Building, Charlotte, N. C.
Architects: Harrison and Abramovitz, New York City; A. G. Odell, Jr., and Associates, Charlotte
Contractor: J. A. Jones Construction Co., Charlotte
Structural Engineering: Severud-Elstad-Krueger, New York City

R/C Duct Floors provide complete flexibility of electrical outlets for the new Wachovia Bank Building. On many other important projects reinforced concrete and R/C Duct Floor construction is providing better structures for less money, maximum fire safety, and low annual maintenance.

A pacesetter in banking architecture, the building itself is a 15-story reinforced concrete structure of "long-span" construction supported on a foundation of 829 concrete and steel piles. The "long-span" construction results in office space which is entirely free of interior columns. The Wachovia Bank Building is the first in the Nation to use prismatic cast stone panels which reflect sunlight, cut down glare, and form an unusual and striking exterior design.

Before you build, investigate this economical and flexible medium of construction. Compare . . . and you will design for Reinforced Concrete.



Write for 16-page R/C Duct Floor Bulletin today!

Concrete Reinforcing Steel Institute
38 S. Dearborn St.
Chicago 3, Illinois

Honorary Members

Announced at ACI Convention

Honorary membership in the American Concrete Institute was conferred on three eminent men in recognition of their outstanding service in the field of concrete or in the work of the Institute at the Award's Luncheon during the ACI 55th annual convention in Los Angeles.

New additions to the ACI honorary membership roster are the late Hardy Cross, Yale University, New Haven, Conn., H. J. Gilkey, Iowa State College, Ames, Iowa, and Eduardo Torroja, Instituto Tecnico de la Construccion y del Cemento, Madrid. This brings to 29 the number so honored.

Hardy Cross

Honorary membership was conferred posthumously on Hardy Cross, professor emeritus of civil engineering, Yale University, who was known internationally for his contributions to civil engineering. His greatest contributions were probably in structural engineering in two methods of analyzing indeterminate structures. These methods are generally known as the column analogy and the moment distribution method. He was coauthor with the late Prof. N. D. Morgan, Sr., of a textbook, *Continuous Frames of Reinforced Concrete*. Outside his own field, he developed a standard method of analyzing flow in water distribution systems.

Professor Cross had recently received the Gold Medal of the Institution of Structural Engineers of England, its highest award, for "outstanding contributions to the science and art of structural engineering." The presentation was only the fourth award of this medal in the 50-year history of the Institution and the second to an engineer not a member of the Institution.

Professor Cross graduated from Hampden-Sydney College in 1902, later receiving degrees from Massachusetts Institute of Technology and Harvard University. An honorary DSc was bestowed on him in 1934 by Hampden-Sydney College.

Professor Cross' outside practical experience began in the Bridge Department of the Missouri-Pacific Railway, St. Louis, in 1908, and bridge valuation for the New York Central Railway in 1911.

Hardy Cross

His first academic post, as a civil engineer at Brown University, came after he received his advanced degree at Harvard in 1911. He served in this capacity until 1918 when he left Brown to establish his own general practice as a structural engineer. He returned to the classroom in 1921 when he accepted the position of professor of structural engineering at the University of Illinois. He left Illinois for Yale in 1937.

Professor Cross was active in engineering society work, having written many important papers, several of which were published in the ACI JOURNAL. He became an Institute member in 1924.

Numerous honors had been bestowed on Professor Cross including the Wason Medal of the American Concrete Institute in 1935 for his paper "Why Continuous Frames?", the Norman Medal of the American Society of Civil Engineers (1933), and the Lamme Medal of the American Society for Engineering Education (1944).

H. J. Gilkey

H. J. Gilkey, professor and formerly head of the Department of Theoretical and Applied Mechanics, Iowa State College, Ames, is a past president of ACI and has long been

H. J. Gilkey

active in Institute affairs, both on technical and administrative committees.

Eminent in the field of research in engineering materials, especially concrete, Professor Gilkey has written several textbooks and manuals, as well as about 150 technical papers and discussions, dealing with concrete, engineering materials, and testing methods. Among his sizable list of published works are the concrete sections of the Urquhart *Civil Engineering Handbook* (3rd and 4th Editions), and the cement products section of the *Encyclopedia of Chemical Technology*. Along with four other ACI colleagues, Professor Gilkey served on the Concrete Research Board for Hoover Dam.

Educated at Oregon State College, Massachusetts Institute of Technology, Harvard University, and the University of Illinois, he began his teaching and research at the University of Illinois in 1921. From 1923 to 1931, Professor Gilkey served in the Civil Engineering Department of the University of Colorado with virtually every nonteaching moment being spent in the laboratory on a variety of one- and two-man reconnaissances.

In 1931, Professor Gilkey was offered the opportunity to organize and head a department of T & AM at Iowa State College, a position from which he retired, administratively, in 1955.

Professor Gilkey was co-Wason medalist of the Institute in 1939 for the paper "The Bond between Concrete and Steel" with S. J. Chamberlin and R. W. Beal. In 1958 he was awarded the ACI Turner Medal for "notable leadership in advancing the knowledge of properties of plain and reinforced concrete." In 1956, he was made an honorary member of both ASTM and its Committee C-9.

Professor Gilkey served as a member of the Institute Board of Direction from 1937-38 and again in 1945-46. He was elected a vice-president in 1947 and president in 1949.

Concurrent and supplementary affiliations include membership in ASCE, American Society for Engineering Education, American Academy of Arts and Sciences, Western Society of Engineers, Society of American Military Engineers, and the Iowa Engineering Society.

Eduardo Torroja

Eduardo Torroja, director, Instituto Tecnico de la Construccion y del Cemento (Technical Institute of Construction and Concrete), Madrid, Spain, received his degree in civil engineering in 1923 and worked in construction until 1927 when he established his own consulting practice. In 1934, he formed the Technical Institute of Construction and Concrete and has been director of it since. The Institute has been referred to as one of the world's three outstanding model testing laboratories. The results of tests at the institute have formed the basis for most of Professor Torroja's own complicated designs.

Eduardo Torroja

Professor Torroja has designed about a thousand projects among which the following are best known: shell roofs for the Algeciras Market Hall, the Recolotes Pelota Hall, and the Zarzuela Horse Racing Track in Madrid; the Esla Viaduct; the aqueducts of Tempul, Alloz, and Tablellina; the bridges of Tordera; the hangars of Torrejon, Barajas, and Cuatro Vientos; the churches of Pont de Suert; and the Fedala elevated water tank.

Professor Torroja has published over 40 papers and books, and lectured throughout the world on structures and concrete.

Professor Torroja has been a member of ACI since 1946, and is president of the International Federation for Prestressing, and the International Committee for Shell Structures. He is a member of ASCE, International Council for Building, the Reunion Internationale des Laboratoires d'Essais des Materiaux et des Constructions, European Concrete Committee, the International Association for Bridges and Structures, and many technical societies in Spain.

LONGEST PRESTRESSED CONCRETE SPAN IN U.S.— *BUILT WITH LEHIGH CEMENT*

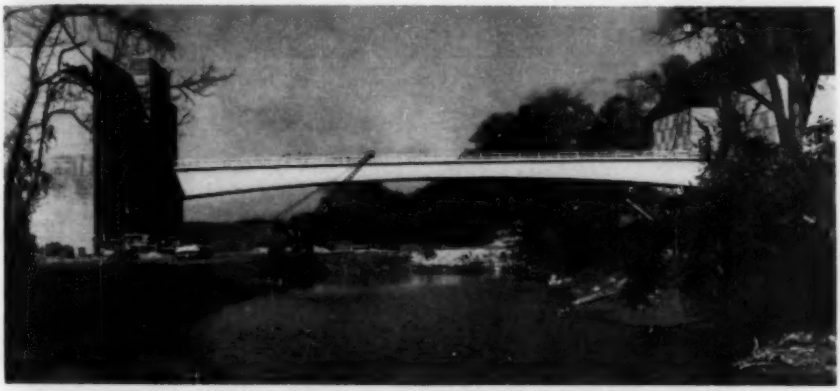
Engineers: Corps of Engineers, U.S. Army, Washington District

Architect-Engineers: Black and Veatch, Kansas City, Mo.

General Contractor: James McHugh Construction Co., Chicago, Ill.

Sub-Contractor for Prestressing: Prestressing Inc., San Antonio, Texas

An extensive false-work system carried formwork for this record *single span* hollow box girder. Huge 700-ton concrete counterweights, which balance the dead load of the bridge, are housed in the two station buildings.



This beautiful *single span* bridge is only 7′ deep at center and 12′ deep at the ends. The roadway on the bridge is 16′ wide. Future plans call for a dual-lane highway running along the C. & O. canal.

• Arching 216′ across the C. & O. canal paralleling the Potomac River at Little Falls, Md., this cast-in-place, post-tensioned Washington Aqueduct bridge joins a riverside pumping station with an electrical sub-station and forms part of the new $6 million water supply project for Washington, D. C.

With Lehigh Cement used throughout, the prestressed concrete span supplants the 160′ center span of the Walnut Lane Bridge in Philadelphia (another "all-Lehigh Cement" project) as the longest in the country.

Once again, in a record-breaking job demanding the best in men and materials, Lehigh Cement has proved its value.

LEHIGH PORTLAND CEMENT CO.
ALLENTOWN, PA.

Note attractive architectural treatment of concrete surfaces of the sub-station. Over 60,000 barrels of Lehigh Cement were used in the entire project which includes the bridge, buildings, a dam and an outlet tunnel.

"Multi-story, Lift Slab Job"

300 Y-S 400 Y-S 500 Y-S

Techkote Company developed the original curing and bond-breaking agent used successfully during the early testing phases of the Youtz-Slick Lift Slab Method of Construction. This compound is known as *Techkote Lift Slab Compound 300 Y-S*. As additional requirements for special treatments became apparent, Techkote Company developed *Techkote Lift Slab Compound 400 Y-S* and *Techkote Lift Slab Compound 500 Y-S*.

It is highly essential in casting these large slabs that maximum strength concrete be produced. Four basic requirements to obtain this are: correct mix-design, thorough mixing, workmanlike placing and proper curing. Techkote Lift Slab Compounds produce this proper curing. They comply with leading Governmental Specifications.

TECHKOTE LIFT SLAB COMPOUND 300 Y-S *Wax-base compound. Provides excellent cure and bond-breaking at low cost. Is effective on all surfaces including floated, broomed, etc.*

TECHKOTE LIFT SLAB COMPOUND 400 Y-S *Wax-free compound. Produces a tough film that provides excellent cure and bond-breaking and is resistant to rainfall and abrasion. Film is compatible with good quality adhesives, paints, etc.*

TECHKOTE LIFT SLAB COMPOUND 500 Y-S *Wax-free compound. Leaves a minimum film that provides good cure and bond-breaking. Film weathers away quickly. Requires a minimum of preparation for painting, placing tile, etc.*

TECHKOTE COMPANY

A DIVISION OF AMERICAN-MARIETTA COMPANY

600 Lairport Street, El Segundo, California

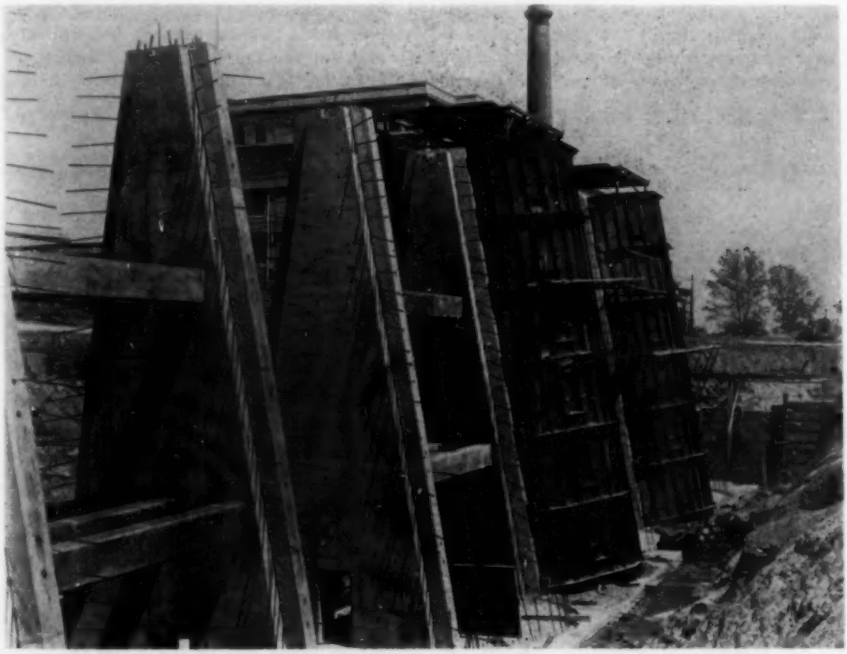
concrete shells preserve water purity

SLOPING CYLINDRICAL CONCRETE SHELLS form the walls for a huge underground clear-water basin where ten million gallons of purified water will be stored at the Chain of Rocks Plant, St. Louis, Mo. Each wall of the square basin, 281 ft long and 30 ft high, is formed of 15 of the hollow concrete cells, interconnected so that inspectors can examine the interior for the entire length of the wall. Every other cell is stabilized with five struts. Struts are omitted from intervening cells so that the arch can flex with expansion or contraction of the structure.

Roof of the basin is a 225 ft square concrete flat slab with tar and gravel upper surface, supported by 225 columns. It will be 3 ft below the ground surface. The roof is free floating, unconnected with the cylinders which form the walls, and thus rides above the cells during expansion and contraction.

Ground water excluded

This patented design, originated by E. E. Easterday chief engineer of the St. Louis Water Department, provides a virtually foolproof barrier against penetration of ground



Courtesy Laclede Steel Co.

FORMS IN PLACE for some of the cylindrical shells that make up the walls of St. Louis water storage basin. Note stabilizing struts that are used in alternate shells. Doorway visible at base of one triangular wall is part of the system that permits inspection of the entire length of the wall's interior

VIEW from the outside of the basin, with forms stripped from one of the 281 ft long, 30 ft high walls

Courtesy Laclede Steel Co.

water into the drinking water supply. An interior drainage system at the roof and ground levels carries away any ground water which might penetrate the cells. The ground level drainage system is porous, to relieve hydrostatic pressures from flood or high water tables.

Some 11,500 cu yd of concrete will be used in constructing the basin, along with 928 tons of multirib, round reinforcing bars and other steel reinforcement. General contractor on the project is G. L. Tarlton, St. Louis. The Water Department of the City of St. Louis is acting as engineer.

. . . honor the construction man

Would your construction job form the basis of an ACI **Journal** paper? Every job superintendent or project engineer keeps a daily account of his job as a regular part of it. Why not use those daily reports, with a few regular progress photos, as the basis for a description of how your firm handles everyday and unusual concreting problems?

Good descriptions of construction projects, or interesting phases of construction projects, are eligible for the ACI Construction Practice Award. One reason for establishing the award in 1944 was to ". . . honor the construction man—the man whose resourcefulness comes in between the paper conception and the solid fact of a completed structure." Another was to enrich the literature of the construction field. The award is given for a construction paper of outstanding merit in the latest **Journal** volume.*

Your paper may be the paper of "outstanding merit" in the next **Proceedings** volume.

PLAIN
1 : 2 : 4 Mix

Durability
Factor: 100

WITH AIR-ENTRAINING AGENT
1 : 2 : 4 Mix

Durability
Factor: 234

WITH PLACEWEL
1 : 2 : 4 Mix

Durability
Factor: 350

Now... from Johns-Manville...

3½ times better freeze-thaw resistance with Placewel,® the liquid water-reducer

The above photos show the results of 21 freeze-thaw cycles in a 5% calcium chloride solution. Under standard A.S.T.M. procedures, Placewel concrete had 3½ times greater resistance than plain concrete and was 1½ times better than straight air-entrained concrete.

Liquid Placewel contains air-entraining and dispersing-water reducing agents, *plus* a catalyst to step up hydrolysis and hydration of cement. By breaking up cement flocs, it releases water for lubrication which would not otherwise be available . . . requires less water in a mix without relying on air-entrainment alone.

You get all the advantages of entrained air without its adverse effects on strength.

Less than 3 oz. Placewel/sack portland cement will—

- increase strength 30% or more (after 28 days' curing)
- improve quality at no extra material cost
- increase workability and placeability
- reduce bleeding and segregation
- give controlled air entrainment
- reduce construction, maintenance costs

For complete technical assistance and the address of our representative nearest you, contact Johns-Manville, Box 14, New York 16, N.Y. In Canada, Port Credit, Ontario.

JOHNS-MANVILLE JM PRODUCTS

JOHNS-MANVILLE

Save days on spring concrete schedules ... add Columbia Calcium Chloride for stronger concrete ... faster!

Chilly days and cold nights can upset your spring pouring schedules. That's why it's important to treat your mix with Columbia Calcium Chloride. This all-weather set accelerator helps you "stay on schedule" three ways:

1. **Faster initial set.** Time is cut from 3 hours to 1 hour under normal conditions. As the temperature drops, Columbia Calcium Chloride's effect is even greater.
2. **Faster final set.** The same ratio holds true; you get final set in just 2 hours rather than the usual 6.
3. **Accelerated early strength.** Columbia Calcium Chloride treatment produces 2 day strength in 24 hours, 7 day strength in 3½ days.

These specific advantages of Columbia Calcium Chloride enable you to finish earlier, pull forms faster. Help eliminate behind-schedule work that sends your costs soaring.

Now is a good time to analyze your batching procedures or check your ready-mix supplier. He can add Columbia Calcium Chloride at his plant or on your job site.

Want more information? Write to any of our fourteen District Sales Offices or to our Pittsburgh address.

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IN CANADA: Standard Chemical Limited

Positions and Projects

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 115, Research

Manuel Rocha
Laboratorio Nacional de Engenharia Civil
Lisbon, Portugal

Committee 623, Foamed Concretes

Cecil W. Foster
Reflectal Corp.
Chicago 4, Ill.

Group from Guatemala visit ACI headquarters

On January 28, six visitors from Guatemala viewed the ACI headquarters building. Mario P. Lehnhoff acted as group spokesman.

CIB of Detroit honors Winkworth

Jack Winkworth of the Winkworth Fuel & Supply Co. was unanimously chosen by the Concrete Improvement Board of Detroit to receive the second annual Huron Achievement Award for his outstanding contributions in promoting concrete quality.

Five years ago Mr. Winkworth was one of the prime factors in the founding of the Concrete Improvement Board, and served as its first chairman.

Currently he is president of the Michigan Ready Mixed Concrete Association, member of ACI and ASTM, and serving on committees of both these organizations. He was one of the speakers on the program of the National Ready Mixed Concrete Association at New Orleans in February.

Concrete Improvement Board directors elected for 1959 were: ACI members Larry Clark of American Aggregates, chairman for 1959; O. R. Bellucci, Giffels and Rossetti; Bill Dreisig, Master Builders; Jack Irmscher of Peerless Cement; Ben Maibach, Jr. of Barton-Malow; and Robert E. Wilde of the American Concrete Institute. Other directors elected were Tom Burke of Cyril J. Burke; John Larson of "Bildor"; and Arthur Moy of the Detroit Department of Buildings and Safety Engineering. Donald H. Ziegler, Cooper Supply Co., was named secretary.

Nordby appointed to Committee on Engineering Sciences

Gene M. Nordby, head of the department of civil engineering at the University of Arizona, Tucson, has been appointed to the Engineers Joint Council Committee on Engineering Sciences.

The Committee on Engineering Sciences is appointed to cooperate with the National Science Foundation and other government agencies in initiating and developing basic research in engineering sciences. Dr. Nordby has also served as program director for Engineering Sciences for the National Science Foundation.

Permanente establishes cement distribution facility on Guam

Permanente Cement Co., Oakland, Calif., has established a cement distribution facility on Guam according to a recent announcement by Wallace A. Marsh, vice-president and general manager.

The plant, located near Apra Harbor on land leased from the Navy, will supply military and civilian construction in the area.

Permanente's new $250,000 plant will have a storage capacity of 60,000 bbl and will be equipped to provide both sacked and bulk cement.

Texas Industries acquires Dallas Lightweight Aggregate

Directors of Dallas Lightweight Aggregate Co. have voted to sell all assets of the company to Texas Industries, Inc., through an exchange of stock.

Dallas Lightweight will continue to do business under the original name and with the same management and personnel.

Texas Industries, Inc. has lightweight aggregate plants in Texas, Louisiana, Oklahoma, and Kansas.

Before you pour

get longer life
for concrete with

CLINTON WELDED WIRE FABRIC

More and more contractors are realizing that it pays to install Welded Wire Fabric wherever concrete construction is called for. They have found through experience that fabric reinforcement is superior in several important ways:

- **Minimizes warping and heaving caused by varying temperatures and moisture content**
- **Distributes shrinkage stresses to minimize cracking while concrete is setting**
- **Should a crack occur, the fabric holds it tightly together, preventing moisture and earth from entering and expanding it**
- **Lengthens life of concrete, preserves its smooth, attractive surface, and keeps maintenance costs to an absolute minimum**

You get all these advantages when you use Clinton Welded Wire Fabric, plus the knowledge that it's a quality product made and backed by a nationwide steel producer. Readily available in both the East and West, in all popular sizes and lengths, Clinton Welded Wire Fabric is your easiest, most dependable way to assure longer life for concrete.

WHEN THEY ASK . . .
"Is it Reinforced"
SAY YES . . . WITH

CLINTON
WELDED WIRE FABRIC
THE COLORADO FUEL AND IRON CORPORATION

5814

THE COLORADO FUEL AND IRON CORPORATION—Albuquerque • Amarillo • Billings • Boise • Butte • Denver • El Paso Fort Worth • Houston • Kansas City • Lincoln (Neb.) • Los Angeles • Oakland • Oklahoma City • Phoenix • Portland • Pueblo Salt Lake City • San Francisco • San Leandro • Seattle • Spokane • Wichita • **WICKWIRE SPENCER STEEL DIVISION**—Atlanta Boston • Buffalo • Chicago • Detroit • New Orleans • New York • Philadelphia • **CF&I OFFICES IN CANADA:** Montreal Toronto • **CANADIAN REPRESENTATIVES AT:** Calgary • Edmonton • Vancouver • Winnipeg

PCA announces appointments to San Francisco office

The Portland Cement Association began field service activities in northern California, northern Nevada, and Oregon effective Jan. 1, 1959, with the opening of a San Francisco office and the appointment of personnel to service the areas.

Charles F. Moran, structural engineering specialist, and Robert E. Jones, paving engineer and general field engineer, both formerly of the association's Los Angeles district office, have been assigned to the San Francisco office.

Byron E. Jones, paving engineer and general field engineer, formerly of the Los Angeles office, will service the state of Oregon. He will headquarter in Portland, under the supervision of the association's Seattle district office.

Glacier Sand and Gravel purchases materials plant

The purchase of Pacific Building Materials Co. and Readymix Concrete Co. of Portland, Ore., was announced recently by the Glacier Sand and Gravel Co., Seattle.

The purchase involves substantially all assets of the two firms, including two sand and gravel plants in Portland, and three ready-mixed concrete plants, two in Portland and one in Vancouver, Wash. Glacier Sand and Gravel is a subsidiary of Permanente Cement Co.

Construction scheduled for New York City Lincoln Center

A group of four of the nation's largest contracting firms has been retained for the construction of Lincoln Center for the Performing Arts in New York City. Preliminary estimates of the cost of the land and buildings of the center total $60 million.

Turner Construction Co., George A. Fuller Co., Walsh Construction Co., and Slattery Contracting Co. have associated in a joint venture to undertake the construction.

Construction of the Center is scheduled to begin this spring with breaking of ground for the new Philharmonic Concert Hall. Other buildings in the Center, scheduled to be completed by 1963, are: the new Metropolitan Opera House; Theater for Repertory Drama; facilities for Juilliard School; Theater for the Dance, and a Library-Museum of the Performing Arts.

Foster and Thomson honored by NRMCA

Alexander Foster, Jr., vice-president of Warner Co., Philadelphia, and Harry F. Thomson, consulting engineer, St. Louis, were presented plaques during the convention of the National Ready Mixed Concrete Association at New Orleans the week of February 16, signifying their election as honorary life members of the board of directors of the association. Both Mr. Foster and Mr. Thomson are former presidents of NRMCA.

Mr. Foster, a member of the Institute since 1912, is a former director of ACI and a past president of the National Sand and Gravel Association.

Mr. Thomson, ACI member since 1928, is past president of ACI and recipient of the 1950 Wason Medal for most meritorious paper of the year, "Specifications Should be Realistic." Mr. Thomson is also a former director of the American Society of Civil Engineers.

Rawn retires

A. M. Rawn has announced his retirement as chief engineer and general manager of the Los Angeles County Sanitation Districts effective Dec. 1, 1958.

Mr. Rawn is currently a member of ACI Committee 210, Resistance to Erosion in Hydraulic Structures.

Ultimate strength design institute held at Marquette

An institute on ultimate strength design for reinforced concrete was conducted at the Marquette University College of Engineering in Milwaukee, Wis., on Monday evenings from January 26-February 23, under the auspices of the civil engineering department and the Portland Cement Association.

Among the speakers were Alfred L. Parme, consulting structural engineer, and Robert J. Van Epps, structural engineer, Portland Ce-

ment Association, Chicago; Phil M. Ferguson, University of Texas, Austin; and Karl Roesser, PCA structural engineer, Milwaukee.

Pomeroy Co. acquires Ben C. Gerwick, Inc.

J. H. Pomeroy & Co., Inc., and Ben C. Gerwick, Inc., recently announced that the Pomeroy company had purchased all of the stock of the Gerwick company. The two companies are heavy construction and engineering firms in San Francisco.

Ben C. Gerwick continues as chairman of the board of that company, and Ben C. Gerwick, Jr., as its president. R. N. Pomeroy and William A. Pomeroy, president and executive vice-president of the Pomeroy company, will be key figures among the directorate of the subsidiary.

Boehmig opens office

Robert L. Boehmig has announced the opening of an office for the practice of structural engineering. Mr. Boehmig will practice in Atlanta, Ga.

Salsbury appointed chief engineer

M. E. Salsbury has been appointed chief engineer of the Los Angeles County Flood Control District replacing H. E. Hedger, who recently retired.

Cawdrey installed as 1959 AGC president

James W. Cawdrey, member of the building firm of Cawdrey and Vemo, Seattle, was installed as 1959 president of the Associated General Contractors of America at its 40th annual convention at Miami Beach, January 19-22. John A. Volpe of Malden, Mass., was installed as vice-president for 1959.

Southern Lightweight Aggregate appoints Jones to new post

Southern Lightweight Aggregate Corp. has appointed George W. Jones, Jr. as representative for the company's new subsidiary, Florida Solite Corp. Offices are in Jacksonville and the plant is located in nearby Green Cove Springs.

Turner Construction announces two executive appointments

H. C. Turner, Jr. chairman of the board and president, Turner Construction Co., recently announced that George E. Horr has been elected vice-chairman of the board, and Francis B. Warren has been elected executive vice-president.

Mr. Horr has served as executive vice-president since 1951. Mr. Warren has served as a vice-president since 1951 and as treasurer since 1953. He will continue as treasurer of the company.

man conquers mountains

And, some day, man will also conquer cancer. With your help.

Guard your family . . . fight cancer with a checkup and a check

AMERICAN CANCER SOCIETY

Lightweight Concrete Skyscraper...

concrete with POZZOLITH*

Designers and builders of fine structures such as this $10-million lightweight concrete skyscraper employ POZZOLITH to obtain unmatched performance and lowest cost-in-place concrete

POZZOLITH is widely used in lightweight aggregate concrete. In this case—the mix weighed not more than 102 pounds per cubic foot. Compressive strength averaged 3750 psi with 6 sacks of cement per cubic yard.

POZZOLITH improves concrete quality and reduces the cost of concrete in place by providing three basic controls: control of water content—makes possible lowest water content for a given workability, control of entrained air, and control of rate of hardening.

Any one of our more than 100 skilled fieldmen will be glad to discuss and demonstrate the benefits of these POZZOLITH controls for *your* projects.

Illuminating Building, Cleveland, Ohio. In-construction and completed views.

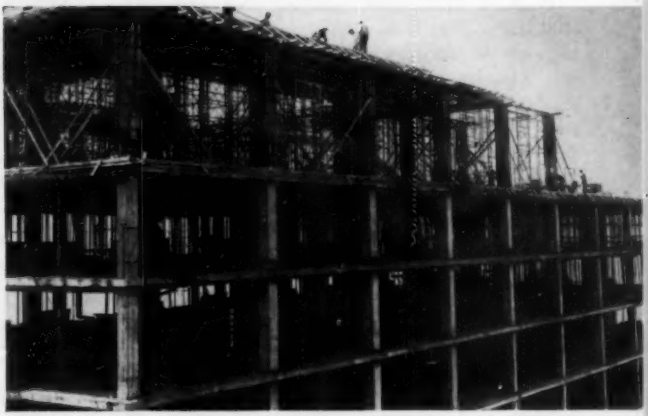
Architects: Carson & Lundin, New York City.

Engineers: McGeorge-Hargett & Assoc., Cleveland.

Contractor: George A. Fuller Co., New York City.

Pozzolith Ready-Mixed Concrete—Cleveland Builders Supply Co.

*POZZOLITH is a registered trademark of The Master Builders Co.



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DIVISION OF AMERICAN-MARIETTA CO
General Offices: Cleveland 3, Ohio • Toronto 9, Ontario • Export: New York 17, N. Y.
Branch Offices in All Principal Cities • Cable: Mastmethod, N. Y.

Hardy Cross

Hardy Cross, professor emeritus of civil engineering, Yale University, New Haven, Conn., died in February. Professor Cross was to have received Honorary Membership in the Institute on February 25. The award was made posthumously.

An Institute member since 1924, Professor Cross had an illustrious career in civil engineering and was a world renowned authority in the field of structural engineering. Perhaps his greatest contributions to that field were two methods of analyzing indeterminate structures; these are generally known as the column analogy and the moment distribution method. He coauthored *Continuous Frames of Reinforced Concrete* with the late Prof. N. D. Morgan, Sr. Outside his own specialty, he developed a standard method of analyzing flow in water distribution systems.

Professor Cross received his BS from Hampden-Sydney College in 1902, and later received degrees from Massachusetts Institute of Technology and Harvard University. Hampden-Sydney College bestowed an honorary DSc on him in 1934. His practical experience began in the Bridge Department of the Missouri-Pacific Railway, St. Louis, in 1908, and bridge valuation for the New York Central Railway in 1911.

Professor Cross' first academic post, as a civil engineer at Brown University, followed his work at Harvard in 1911. He served in this capacity until 1918 when he left to establish his own practice as a structural engineer. He returned to the classroom in 1921 as professor of structural engineering at the University of Illinois. He left Illinois for Yale in 1937.

Professor Cross wrote many technical papers in his long career, several of which were published in the ACI JOURNAL. In 1935 he was awarded the Wason Medal for the year's "most meritorious paper" for the contribution "Why Continuous Frames?" Among the recognitions of his work were the Norman Medal of the American Society of Civil Engineers (1933), the Lamme Medal of the American Society for Engineering Education (1944), and the Gold Medal of the Institution of Structural Engineers of England (1958). This latter award for "outstanding contributions to the science and art of structural engineering," was only the fourth award of this medal in the 50-year history of the Institution and the second to an engineer not a member of the Institution.

Concrete conference conducted at Utah State University

The civil engineering department, Utah State University, Logan, conducted a concrete conference on February 20 with William A. Cordon serving as conference director.

Phases of concrete covered at the various sessions included curing, quality control, specifying, joints and crack control, and concrete construction on the farm.

A film showing the construction of the new American Concrete Institute Headquarters Building was shown with C. Pickett, structural engineer, Portland Cement Association, Salt Lake City, Utah, serving as narrator.

Texas Industries enters field of decorative panels

Texas Industries, Inc., Dallas, has announced its entry into the field of decorative architectural panels through the acquisition of the Texas-Louisiana division of Wailes Precast Concrete Corp. according to a joint announcement by Ralph B. Rogers, president of Texas Industries, and C. D. Wailes, Jr., president of the Wailes Co. In the transaction Texas Industries acquired Wailes' Texas plant and the rights to manufacture and sell Mo-Sai decorative architectural products in Texas and Louisiana.

The Wailes precast plant at Los Angeles is not involved in the transaction.

Sika Chemical announces promotion of two key engineers

The promotion of two engineers on Sika Chemical Corp. staff and the appointment of a third was recently announced by Dr. Emil Schmid, president.

R. A. Jessen becomes chief engineer; J. Wayman Williams, Jr., assumes the key position of manager of technical service. Robert K. Lockwood joins Sika as advertising manager after 10 years on the staff of the American Society of Civil Engineers.

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. The grades of membership are described overleaf.

All who have an interest in concrete are eligible for membership.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI JOURNAL provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

Opportunity for service is present in technical committee activity, in contributions, or only comments, to the ACI JOURNAL, or in reviewing technical publications for material of interest to the membership.

ACI's world-wide membership is growing in extent and participation—traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

(cut here)

Board of Direction, American Concrete Institute **Date**_____
P. O. Box 4754, Redford Station
Detroit 19, Michigan

Individual Members (U. S. and Possessions, Canada, Mexico, Central America, and West Indies)	**$15.00**
Individual Members (All other foreign countries)	**12.00**
Corporation Members	**50.00**
Contributing Members	**100.00**
Junior Members—nonvoting (under 28)	**7.50**
Student Members—nonvoting (under 28)	**5.00**

(Subject to stipulations of Bylaws—Article I on reverse side. Bylaws on request.)

Of the annual dues, $12.00 is for the JOURNAL of the American Concrete Institute (except that dues for Junior and Student Members apply in full for the JOURNAL).

The undersigned hereby applies for _____ **membership**
(Individual, Corporation, Contributing, Junior, Student)

in the American Concrete Institute. Proposed by_____

For Corporation Membership, ACI representative will be_____

_____ _____
(Date of graduation if Student) (Name, if Corporation)

Signature_____

For our records, please complete both sides of application.

EXCERPTS FROM BYLAWS: ARTICLE I—MEMBERS

Section 1. This Institute shall consist of Honorary Members, Corporation Members, Contributing Members, Members, Junior Members and Student Members.

Sec. 3. A Member shall be a person.

A Corporation Member shall be a firm, corporation, society, agency of government, or other organization.

A Contributing Member shall be a person, firm, corporation, society, agency of government, or other organization electing to give greater support to Institute activities through the payment of larger dues. Any Contributing or Corporation Member, other than a person, may name a personal representative who shall enjoy all membership rights and privileges.

A Junior Member shall be a person less than 28 years of age.

A Student Member shall be a person less than 28 years of age and a registered student at a technical or engineering school.

Sec. 4. All classes of Members, except Honorary Members and Student Members, shall be sponsored by at least one Member of the Institute. An Honorary Member shall be elected by unanimous vote of the Board of Direction. A Student Member shall be sponsored either by a Member of the Institute or by a member of his school's faculty, who need not be a Member of the Institute.

Sec. 5. All Members in any classification shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall neither vote nor hold office. The status of a Student Member shall change automatically to that of Junior Member or Member, depending on age, on the first anniversary of his membership succeeding the date on which he ceases to be a registered student. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

Sec. 6. Applications for and resignations from membership and requests for change of representatives of Corporation or Contributing Members shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from Members whose dues are not more than 60 days in arrears, except by special action of the Board of Direction.

(cut here)

Name _____
Please Print

Date of Birth _____
Month Day Year

Title or Position _____

Name of Firm or Organization _____

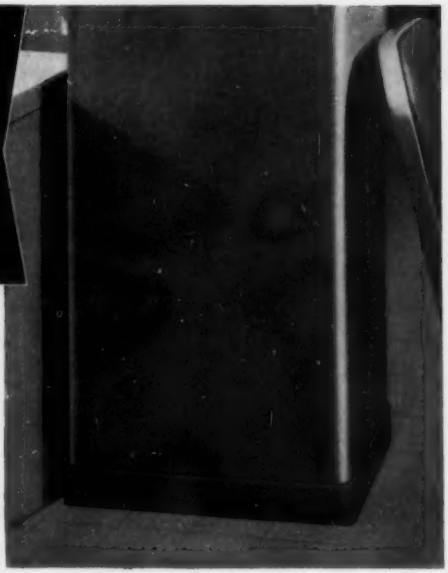
☐ **Business Address** _____

☐ **Resident Address** _____
(Please check address to which you wish mail and publications sent)

Nature of Firm's Business _____

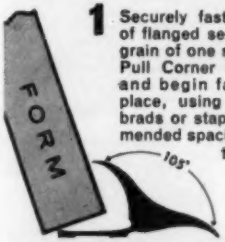
The ACI Membership Directory will be sent—as available—only on request. Check here if you wish to receive the latest edition. ☐

FOR *Rounded Concrete Corners*



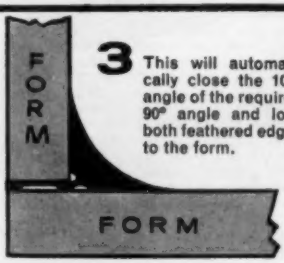
SERVICISED re-usable "GREEN STREAK" Corner Former

SIMPLE INSTALLATION



1 Securely fasten top end of flanged section to end grain of one side of form. Pull Corner Former taut and begin fastening in place, using strip nails, brads or staples. Recommended spacing not more than 4" apart.

2 After Corner Former is fastened, place next side of form in position and tie by conventional methods to form the corner joint.



3 This will automatically close the 105° angle of the required 90° angle and lock both feathered edges to the form.

Servicised "Green Streak" Corner Former provides a quick, easy and low cost method of forming perfect 1" radius rounded corners on piers, beams and all outside corners of poured concrete. Made of a tough, durable and resilient plastic, the Corner Former is simply fastened to the form (see installation drawings at left) and it automatically assumes the correct radius. Because it is readily removed and re-installed, Servicised "Green Streak" Corner Former is by far the lowest cost method of round corner forming you can use.

Available in standard lengths of 10 ft. Four 10 ft. pieces packaged in a tube.

Write for full details and prices.

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SERVICISED PRODUCTS CORPORATION

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Honor Roll

January 1—February 28, 1959

PUT YOUR NAME ON THE "HONOR ROLL" IN '59. Expand ACI by sponsoring new members who will benefit through affiliation with YOUR Institute.

David A. Sauer	5
Robert P. Witt	4½
Rafael Enrique Pacheco	3
John P. Thompson	3
John E. Breen	2
Boris Bresler	2
W. S. Cottingham	2
Phil M. Ferguson	2
Joachim F. Leppmann	2
Charles C. Luther	2
Ferruh Taskin	2
A. M. Tovar	2
Jose R. Perez Valentin	2
C. K. Wang	2
Roberto Zepeda Aldana	2

New Members

The Board of Direction approved 80 Individual applications, 2 Corporation, 21 Junior, and 13 Student applications. Considering losses due to deaths, resignations, and nonpayment of dues the total membership on February 1, 1959 was 9764.

Individual

ALLEN, WILLIAM B., San Francisco, Calif. (Design Engr., Guy F. Atkinson Co.)

AVANT, J., Albuquerque, N. M. (Struct. Engr., Chf. of Civil Works Sub-section, Albuquerque Dist. Corps of Engrs.)

BERNARD, RICHARD V., Detroit, Mich. (Assoc. C. E., D. P. W., City of Detroit, Bur. of Test & Insp.)

BHARATHAN, KONNANATT, Calcutta, India (Exec. Engr., Pub. Works Dept., Gov't of Kerala State)

BONINI, FRANCIS J., Middletown, N. Y. (Supv. of Constr., N. Y. State Dept. of Pub. Wks.)

BRIGHT, FRANK, Ajax, Ontario, Canada (Struct. Engr., Wilson & Newton, Archs.)

BRUCKY, IVAN, Ottawa, Canada (Constr. Engr., Corp. of the City of Ottawa)

CONLEY, ARTHUR, Olathe, Kan. (Chf. Draftsman, C. W. Hofsinger Co.)

CRAWFORD, EDGAR G., San Marina, Calif. (Constr. Supt., Ben F. Smith, Inc.)

DELANEY, CARROLL S., Denver, Colo. (Cons. Engr., Struct.)

EHRET, LE ROY G., Los Angeles, Calif. (Partner, Neptune Mfg. Co.)

EPSTEIN, RAYMOND, Chicago, Ill. (Pres., A. Epstein & Sons, Inc.)

ESTES, F. DOUGLAS, Saint Albans, W. Va. (Sales Repres., Master Builders Co.)

FIGUERO, JULIO ARENAS, Incapuquio, Tacna, Peru (Asst. Area Engr., Proyecto de Toquepala-Cia. Utah Pacific Co. and M. K.)

FORRER, RUDOLPH F., Milwaukee, Wis. (Pres., Spray-O-Bond Co.)

FREEMAN, JOHN WILLIAM, Atlanta, Ga. (Struct. Engr., Armstrong & Edwards)

GLYNN, WALTER S., Toronto, Canada (Struct. Engr., Marani & Morris, Archs.)

GOLDING, J. B., Rose Bay, Australia (Director & Gen. Mgr., Cement Chemicals (Aust.) Pty. Ltd.)

GOODEN, JOSE A., Lagunillas, Estado Zulia, Venezuela (Projects & Insp., Instituto Nacional de Obras Sanitarias)

GOODRICH, D. E., Adrian, Mich. (Partner, Lenawee Constr. Co.)

GORDON, STANISLAW R., Jackson Heights, N. Y. (Sr. Designer in Conc. Hydr. Design Dept., Ebasco Services, Inc.)

GRAY, GEORGE H., Chicago, Ill. (Chem. Engr., Prod. Dev., Enterprise Paint Co.)

GUILLEMETTE, JOSEPH D., Providence, R. I. (Partner & Sr. Engr., J. D. Guillemette & Assocs.)

GUTIERREZ, MANUEL, Ann Arbor, Mich. (Grad. Stud. U of M)

HINDS, CLAUDE D., Mesquite, Tex. (Struct. Engr., Edward L. Wilson, Jr.)

JOHNSON, WARREN W., Kalamazoo, Mich. (Pres., Kalamazoo Ready Mix Concrete Co.)

KEETON, JOHN R., Port Hueneme, Calif. (Sr. Proj. Engr., U. S. Naval Civil Engrg. Lab.)

KEHART, MARTIN W., Washington, D. C., (Inspector General, U. S. Navy Dept.)

KESSLER, LLOYD L., Waukesha, Wis. (Struct. Engr., Schutte-Phillips-Mochon)

KISCHLAT, WALTER L. H., Long Island, N. Y. (Struct. Design, Jos. Neufeld, Arch.)

KLUVERS, BERT, North Vancouver, Canada (Struct. Design & Proj. Engr., A. C. Milroy & Assocs.)

KOKTURK, ALEV, West Newton, Mass. (Proj. Engr., Crandall Dry Dock Engrs., Inc.)

KONOPKA, DONALD J., Pittsburgh, Pa. (Constr. Engr., Insp. & Supv., The Lummus Co.)

KOUTSODEMETROPOULOS, DEMETRIOS, Athens, Greece (Chf. Struct. Engr., Frank E. Basil Co., Inc.)

KRIEVS, ALBERT, Montreal, Canada (Struct. Designer, Canadian National Railways)

KUCHNICKI, DONALD J., Alpena, Mich. (Asst. Gen. Mgr., Admin., prod., & sales, Elastizell Corp. of America)

LA BYER, JOSEPH R., Seattle, Wash. (Struct. Designer, Tracey, Cook & Brunstrom)

LAURIE, JOHN JAMES, Alhambra, Calif. (Design Engr., Consolidated Western Steel Div., U. S. Steel Corp.)

LENHART, JAMES M., II, Los Altos, Calif. (C. E., Carroll E. Bradberry & Assocs.)

LONG, FRENCH S., Detroit, Mich. (Dist. Mgr., Sales & Engrg. Service, Sika Chemical Corp.)

LONGWORTH, JACK, Edmonton, Canada (Assoc. Prof. of Civil Engrg., Univ. of Alta.)

MACDERMOTT, ROBERT W., Kirkwood, Mo. (Struct. Engr.)

MAGNUSSON, L. R., Oxford, Ind. (Struct. Engr., E. R. Hamilton & Assoc.)

MANBER, IRVING, San Francisco, Calif. (C. E., Bechtel Corp.)

MCKEON, RICHARD D., Cleveland, Ohio (Engr., Design & Field Supv., The Masters & Mullen Constr. Co.)

MEIRI, YEHUDA, Rego Park, N. Y. (Struct. Engr., Designer, James Ruderman, Cons. Engr.)

MERTES, CHARLES FREDERICK, San Francisco, Calif. (Asst. Bridge Engr., Calif. Div. of Hwys.)

MORREO H., FELIX ANTONIO, Cabimas, Estado Zulia, Venezuela (Land Engrg. Unit Supv., Creole Petroleum Corp.)

MORRIS, GENE R., Tucson, Ariz. (Field Engr., PCA)

MOTT, FRED L., Boston, Mass. (Partner, Gaudette & Mott)

MURPHY, WILLIAM, Toronto, Canada (Struct. Designer, Kappele, Wright & Macleod, Cons. Engrs.)

NAPIER, BAXTER W., JR., Anchorage, Ky. (C. E., Surveying, Sanitary, Struct.)

NEBEN, JERRY, Far Rockaway, N. Y. (C. E., Supv. of Constr., New York City Transit Authority)

NICHOLSON, LEO P., Northbrook, Ill. (Railway Repres., PCA)

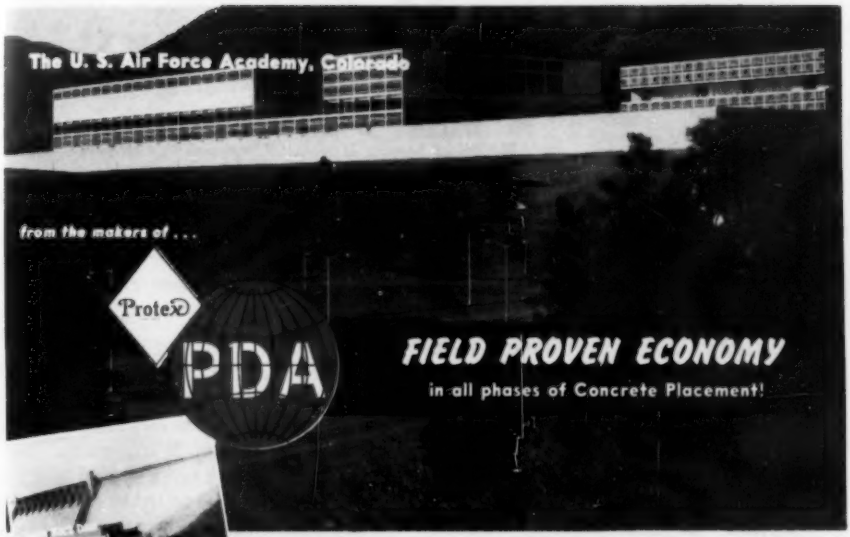
NIGHTINGALE, WILLIAM EDWARD, Wellington, New Zealand (Mfr., Concrete Admixtures, Colored Cements, Cement Paints, PVAC Paints)

OETZEL, EDGAR A., Pasco, Wash. (Constr. Insp., Civil, General Electric Co.)

PARTHASARATHY, VARADACHARDI, Urbana, Ill. (Design Engr., Clark, Daily & Dietz, Cons. Engrs.)

PETTIT, SAM LAYTON, Spartanburg, S. C. (Constr. Engr., Supv., Cecil's, Inc.)

PLAZA, FELICIANO REYNA, Caracas, Venezuela (Chf. Engr., Exploraciones Voladuras y Drenajes, S. A.)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FIELD PROVEN ECONOMY
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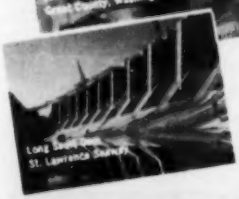

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Darex Diary

by Mel Prior
Member, American Concrete Institute
Technical Service Manager, Construction Products
W. R. GRACE & CO.
Dewey and Almy Chemical Division
(No. 12 of a series)

Solving the problems of hot weather concreting

As in the past, this column will be devoted to a subject of interest to all of us who deal in concrete. Since the Bureau of Public Roads recently initiated a program to study the advantages offered by the use of retarders in hot weather concreting, industry-wide interest has been focused on this subject. Although the term "retarding agents" is itself a familiar one, I would judge from the large number of inquiries I've been receiving that not too much detailed information has been disseminated concerning their purpose, function, and applications.

"Retarding agents," as the term is being used today, does not have reference to gypsum which is one of the earliest forms of a retarder known. Gypsum is used to prevent "flash set" and does not retard the normal setting of cement. Initial retarders slow down the early rate of hydration which is unaffected by gypsum and are not designed to control "flash set."

The purpose, then, of a modern-day initial retarding agent is to eliminate the problems that normally stem from hot weather concreting: the crazing and cracking that occurs from too rapid setting when a normal water-cement ratio is employed; the loss of strength resulting from increasing the water-cement ratio in an effort to avoid the former problem. These problems stem from the fact that summer's intense heat accelerates the hydration process . . . increases the demand of cement for water . . . causes the cement to set before placing is complete.

The function of a modern-day initial retarding agent is to hold off this accelerated demand for water . . . to slow down the hydration process . . . to keep the concrete plastic until placing is complete while at the same time actually *reducing the water-cement ratio as much as eight to ten percent.*

Its applications are manifold. The general rule to follow is that an initial retarding agent should be employed in *any* hot weather concreting where control of the rate of setting is important to the finished job.

In other words, *any* hot weather concreting that demands delayed setting . . . yet must attain maximum strength in the finished concrete . . . demands an initial retarding agent. Other applications for retarding agents will be discussed in subsequent diaries.

Here at Dewey and Almy, we have conducted research and development programs on retarding agents for over twenty years. For free detailed information on how a retarding agent such as DARATARD can improve *your* hot weather concreting, write to:

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LOOKING AHEAD

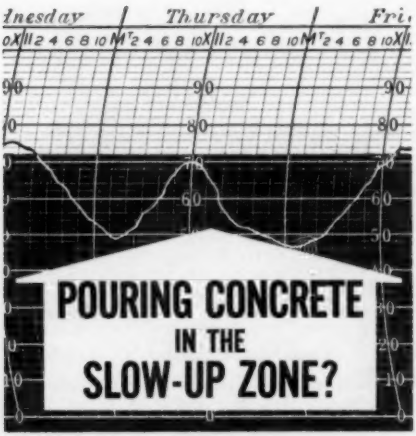
Mar. 11-14, 1959—51st Annual Convention, American Concrete Pipe Association, Palm Beach Biltmore, Palm Beach, Fla.

Apr. 5-9, 1959—1959 Nuclear Congress: an assembly of the 7th Hot Laboratories and Equipment Conference, 5th Nuclear Engineering and Science Conference, 7th Atomic Energy Management Conference, and Atomfair. Engineers Joint Council, Public Auditorium, Cleveland, Ohio

Apr. 6-8, 1959—8th Annual Meeting, Building Research Institute, Penn-Sheraton Hotel, Pittsburgh, Pa.

May 13-15, 1959—"Deutsche Betontag 1959," German Concrete Association, Congress Hall of the German Museum, Munich, Germany

May 18-19, 1959—39th Annual Meeting, The Society of American Military Engineers, Washington, D. C.



Have your ready-mix supplier add SOLVAY CALCIUM CHLORIDE to avoid costly delays!

When temperatures vary from 70° down to 50°—it spells "SLOW-UP" which means costly delays in your concreting operations. This drop below 70° sharply decreases strength development and lengthens the waiting period before finishing.

A drop from 70° to 50°, for example, cuts 3-day strength up to 40%. To prevent this, add a low-cost 2% of SOLVAY Calcium Chloride to your concrete.

With this acceleration, concrete poured at 50° has a 3-day strength up to 40% *greater* than ordinary concrete cured at the ideal temperature of 70°!

When you use SOLVAY Calcium Chloride, you do away with overtime finishing, delays in form removal, delays between operations. You save up to 50% on protection time. And you get *better* concrete—increased in *both* early and ultimate strength—with lower water-cement ratio for more moisture-and-wear-resistant concrete.

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Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Pressurized tank-truck for dry bulk materials

A pressurized tank-truck transport by Delta Tank Manufacturing Co. is equipped with a low-pressure blower which automatically forces bulk cement or other pulverized cargo into storage silos or batch plants through a 4-inch rubber hose. This, according to Delta, eliminates the need for standard type bucket, belt, screw or other auxiliary handling mechanisms at the unloading sites.

Outwardly the unit resembles the standard type of highway tank-truck such as is used for cross-country shipment of large quantities of milk, petroleum products, and other fluids. Available in various sizes to comply with individual state weight and size specifications.

Air for the transport is supplied either by a power-takeoff driven blower on the tractor or from an independently driven blower unit. Effective pumping range is approximately 150 ft in any direction from the discharge control valve at the center of the trailer according to company spokesman.—Delta Tank Mfg. Co., Inc., Baton Rouge, La.

Vibrator

Pacific Mercury's one-man operated concrete vibrator uses low amplitude, high frequency vibration such as is used in sound equipment and is said to be capable of vibrating to depths to 44 ft.

Motor is in the head of the vibrator; there are no bearings in the vibrating device, thus reducing service and maintenance problems according to the manufacturer.

Vibrator is available in two basic motor sizes with either 1¾-in. or 2¾-in. diameter head; casing is available in 7, 14, or 21 ft.—Pacific Mercury Construction Equipment Division, 14052 Burbank Blvd., Van Nuys, Calif.

Concrete batching units with drop beams

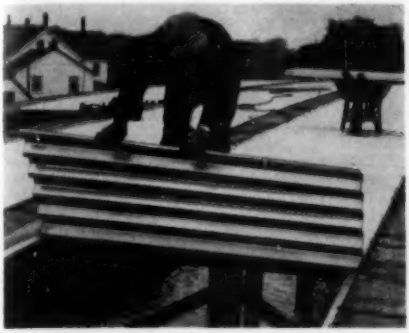
Three concrete batching units with drop beams have been announced by Toledo Scale Corp.: one provides manually-operated drop beams; one automatic cutoff for each ingredient and manually-operated drop beams; and the third motor-operated drop beams.

Each of the three drop beam models uses the Toledo double pendulum dial to provide an accurate weight indication. The motor-operated drop beam system automatically controls all ingredients except water while the other models require either manual cutoff or manual initiation of the feed cycle.

All models in the Toledo line may provide for transmission of weight figures and data to remote locations. Weight data can be typewritten, numerically listed and totaled, stored for delayed recording, recorded by tape punch code, or introduced into a data processing system.—Toledo Scale, Division of Toledo Scale Corp., Toledo 12, Ohio

Galvanized steel forms

Permanent galvanized steel forms for concrete bridge floors are now available through distributors of the Granco Steel Product Co. Photo shows workman



placing Stay-In-Place bridge forms for the rebuilding of the three-lane vehicle deck of the MacArthur Bridge across the Mississippi River, St. Louis.

Available in 20, 18, and 16 gage thicknesses, remain in place after concreting. As each form unit is set in place between stringers, it provides a working platform, ready for reinforcing steel and concrete.—Granco Steel Products Co., 6506 North Broadway, St. Louis 15, Mo.

Column form clamps

Econ-O-Clamps for forming square or rectangular columns are being marketed by Deslauriers Column Mould Co. A hammer is the only tool needed and there are no accessories or separate parts. Clamps are also used in forming templates.

Clamps are manufactured of 2 x 2 x ⅛-in. steel angles, electrically welded into a 90-deg angle; weigh 14 lb per set, and are coated to prevent rusting. Illustrated literature available.—Deslauriers Column Mould Co., 5036 West Lake St., Chicago 44, Ill.

Two-in-one anchor-drill

Star Selfdril Shield, a rustproof masonry anchor that drills its own hole and secures fixtures to masonry, has been announced by Star Expansion Industries Corp.

The 888 two-in-one anchor-drill has eight wedge-shaped teeth which cut into masonry materials without choking, producing a hole invariably clean, accurate and close-fitting according to the manufacturer.

Available in four types and a wide range of sizes for light-to-heavy duty bolt fastening.—Star Expansion Industries Corp., Mountainville, N. Y.

Curing blanket

Fiberglas concrete curing blanket, designed to protect freshly placed concrete while curing has been introduced by Owens-Corning Fiberglas Corp. It is a fine-fibered, flexible, resilient, felt-like blanket of fibrous glass bonded with a thermo-setting resin, completely enclosed in an 8 mil black polyethylene film.

Available in two thicknesses: standard 1 in. and heavy duty 2 in., in a width of 72 in. and a length of 50 ft. Manufacturer states it is easily and quickly put in place, because of its light weight and is specifically designed for repeated use.—Owens-Corning Fiberglas Corp., Toledo 1, Ohio

Resin adhesive

Uniweld, a structural bonding agent for joining fresh concrete to cured concrete, has been introduced by Permagile Corp. of America.

Agent is an alloy of epoxy and nylon-type synthetic resins which forms a permanent joint, and water and vapor barrier, bonding the contact area without any mechanical interlocking according to Permagile spokesman. The bond, which is about 0.010 in. thick, is said to develop its full strength faster than the newly placed concrete.

Also provides adhesion to brick, stone, cinder block, gypsum block, and other commonly used building materials, it is claimed.—Permagile Corp. of America, 37-23 Thirty-third St., Long Island City, N. Y.

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Portable masonry drilling unit

Portable, high speed, heavy duty drilling machine, the Di-Cor, drills clean, perfect holes through reinforced concrete, marble, granite, cement, brick, tile, stone, glass, and other hard, brittle, nonmetallic materials, according to the manufacturer.

The unit is equipped with a 2¼-hp electric drill motor and an integral water swivel which supplies water to the cutting face of a diamond drill to cool the bit and flush away cuttings; wheel-mounted stand makes unit flexible. Manufacturer states the Di-Cor is simple to operate and can be swung through 360 deg to drill in any position at any angle.

Diamond bits, ranging in size from ¼ to 14 in. in diameter, are available.—Diamond Core and Saw, Division of Portomag, Inc., 1521 East Nine Mile, Detroit 20, Mich.

Floor surfacing and patching compound

H-F Epoxy-Crete, a floor surfacing and patching compound developed by Howe & French, Inc., consists of epoxy resin plastic, a hardening agent, and kiln-dried sand. When blended, the mixture bonds to old or new concrete, wood or metal floors, forming a firm, smooth surface that provides resistance to heavy impact, abrasion, and chemical action according to the manufacturer. Manufacturer recommends the compound for walls and floors of food and drug processing plants, as it is not affected by scrubbing with strong alkali soaps and detergents.

Company also states that the compound sets in from 12 to 24 hr and it is suitable for both interior and exterior use.—Howe & French, Inc., Weymouth, Mass.

Hydraulic center-hole rams

Star Jack Co. has extended its line of hydraulic center-hole rams to include capacities of 30 to 300 tons and to provide strokes in some models up to 36 in.

Connected with hydraulic pumps these rams are designed to serve as standard jacks for lifting or pushing loads and also to pull out and replace gears, wheels, propellers, bushing, cylinder liners, shafts, etc., according to the company.—Star Jack Co., Inc., 2638 Davisson St., River Grove, Ill.

Parting compounds

Two parting compounds for reinforced and prestressed concrete forms have been announced by Swift & Co.

Parting Oil No. 833 is a solvent-type material designed for quick-drying and Parting Oil No. 842 is an economical, oil-type compound; both products contain anti-rust additives. They can be applied to forms by spray, brush, or wiping methods.

Manufacturer is now offering the new parting compounds on a trial order basis. Details on request.—Swift & Co., Technical Products Dept., 1800 165th St., Hammond, Ind.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested direcly from the manufacturers listed below.

SINGLE STRAND TENSIONING FOR PRESTRESSED CONCRETE—Folder explains the advantages of pulling single strands over the pulling of multiple strands all at once. Also contains description of single strand jacking unit including complete specifications and price.—Prestressed Equipment Co., P. O. Box 1264, Lakeland, Fla.

BRIDGE DECK SURVEY—Survey covering bridges in the United States and Canada whose construction incorporated expanded shale lightweight concrete. Survey indicated that some 40 structures had been built and a tabulation of the data was prepared and published in 1954. Since then about 30 additional structures have been reported and this information has been incorporated in the tabulation.—Expanded Shale, Clay and Slate Institute, National Press Building, Washington, D. C.

QUALITY CONCRETE WITH CALCIUM CHLORIDE—A 16-mm color and sound motion picture on use of calcium chloride with portland cement concrete. The 12-mm film is available for showing to ready-mixed concrete associations, contractors, colleges and universities, civil and structural engineers, architects, and other technical groups.—The Dow Chemical Co., Midland, Mich.

YEAR ROUND CONCRETING—Eight-page pamphlet summarizes the American Concrete Institute's standard recommendations for cold weather concreting. Explains how calcium chloride and other developments aid in placing durable, quality concrete in cold weather. —Calcium Chloride Institute, 909 Ring Bldg., Washington 6, D. C.

PRELOAD PRESTRESSED CONCRETE TANKS (Bulletin T-22)—Covers history of prestressed concrete tanks. Bulletin gives details on design, construction, cost estimating, and maintenance of prestressed concrete tanks.—The Preload Co., Inc., 211 East 37th St., New York 16, N. Y.

MANUAL OF ACCIDENT PREVENTION IN CONSTRUCTION—Fourteen sectional "pocket-size" reprints from The Associated General Contractors of America *Manual of Accident Prevention in Construction*. Available booklets are: A—Scaffolding and Ladders; B—Explosives and Powder Actuated Tools; C—Housekeeping and Sanitation and First Aid; D—Welding and Cutting; E—Flammable Gases and Liquids, Handling and Storage of Materials and Equipment Upkeep; F—Excavation and Shoring, Barricades, and Pipelines; G—Pile Driving and Marine Equipment; H—Concrete

Construction, Masonry, and Steel Erection; J—Hoists, Cranes and Derricks; K—Highway Construction; L—Heavy Equipment, Motor Trucks, Garages, and Repair Shops; M—Tunnels and Compressed Air Work; O—Inspection Check List; and R—Introduction. Booklets may be purchased individually for 15 cents per copy; complete manual, $3.75 per single copy.—The Associated General Contractors of America, Inc., 20th and E St., N.W., Washington 6, D. C.

STANDARD FOR SAFEGUARDING BUILDING CONSTRUCTION OPERATIONS (NFPA No. 241)—This edition supersedes all previous editions of this Standard; 35 cents. STANDARD METHODS OF FIRE TESTS OF BUILDING CONSTRUCTION AND MATERIALS (NFPA No. 251)—This Standard was adopted in its present revised form by the National Fire Protection Association, May, 1958; 50 cents. METHOD OF TEST OF SURFACE BURNING CHARACTERISTICS OF BUILDING MATERIALS (NFPA NO. 255)—Supersedes the Standard adopted in 1955; 35 cents. All Standards available at discount for quantities.—National Fire Protection Association, 60 Batterymarch St., Boston 10, Mass.

HOW TO GET BETTER CONCRETE PRODUCTS AT LOWER COST—Folder briefly describes uses of Solvay Calcium Chloride in concrete products.—Solvay Process Division, Allied Chemical Corp., 61 Broadway, New York 6, N. Y.

CONCRETE TESTING MACHINE CATALOG—Complete catalog of laboratory, plant, and job-site testing machines with collateral equipment for testing concrete pipe and drain tile, cylinders, blocks, cubes, beams, and lintels.—Forney's Inc., Tester Division, P. O. Box 310, New Castle, Pa.

FLEXICORE FACTS NO. 82—Features construction of the new headquarters building of the American Concrete Institute. Photographs and line drawings describe architect's use of the hollow cells in Flexicore precast floor for heating, cooling, ventilating, and for underfloor electrical distribution.—The Flexicore Co., Inc., 1932 E. Monument Ave., Dayton, Ohio

PRESTRESS STRAND BROCHURE—Describes the properties and characteristics of 7-wire high-strength stress-relieved stand for prestressed concrete. Contains typical stress-strain curves of 3/8- and 7/16-in. strand together with data concerning breaking strength, yield strength at 1 percent elongation, ultimate elongation in 24 in., and typical elongation in 10 ft at recommended prestressing load.—Leschen Wire Rope Division, H. K. Porter Company, Inc., 2727 Hamilton Ave., St. Louis 12, Mo.

FULLER EQUIPMENT FOR THE PROCESS INDUSTRIES (Bulletin G-38)—Comprehensive 12-page, two-color bulletin describes Fuller equipment for the process industries. Photographs, drawings, schematic illustrations, and tables of dimensions and capacities supplement text.—Fuller Co., Catasauqua, Pa.

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BUILDING A HIGHWAY—A 16-mm sound and color motion picture explaining, in layman's language, the building of a modern highway. Depicts important phases of highway construction, explains planning, engineering skill, and economic considerations behind them. For use primarily before civic, social, business, and technical organizations.—Portland Cement Association, 33 West Grand Ave., Chicago 10, Ill.

UNITED STATES TESTING CO. SERVICES AND FACILITIES (Bulletin 5801)—Describes the complete line of laboratory and field testing facilities and services available from United States Testing Co., including description of the company's engineering facilities and services.—United States Testing Co., Inc., 1415 Park Ave., Hoboken, N. J.

POZZOLITH IMPROVES CONCRETE QUALITY FOR AIRPORT PAVEMENTS AND STRUCTURES (Master Builders Reporter No. 15)—Twenty-page publication discusses problems encountered in concreting airport pavements and structures. Job story reports cover nine important airport projects in the United States, Canada, and the Dominican Republic. Cite role played by Pozzolith in obtaining the high quality concrete required for this type of work.—The Master Builders Co., 7016 Euclid Ave., Cleveland 3, Ohio

HYDROCIDE COLORCOAT (No. BP 3063)—Illustrated brochure outlining the properties and advantages of Hydrocide Colorcoat, a heavy-bodied, textured decorative coating for exterior and interior masonry walls above grade. Color chart included.—L. Sonneborn Sons, Inc., Building Products Division, Dept. H, 404 Fourth Avenue, New York 16, N. Y.

TRANSLATION PUBLICATIONS—1959—A 28-page catalog of translations of Russian and Czechoslovakian scientific publications.—Consultants Bureau, Inc., 227 W. 17th St., New York 11, N. Y.

BULK WEIGHING SCALE (Product Data Sheet 5807)—Data sheet describes the Richardson Model MSM scale for bulk weighing of dry and free-flowing as well as sluggish materials. Explains how the scale weighs and records mechanically the weights of materials ranging from light hulls to heavy ores. Photograph, drawings, and specifications of the unit included.—Richardson Scale Co., Clifton, N. J.

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DISCUSSION, PROCEEDINGS V. 55

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